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TECHNICAL REPORT REMR-CS-6

IN SITU REPAIR OF DETERIORATED CONCRETE IN HYDRAULIC STRUCTURES: FEASIBILITY STUDIES

by

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	Problem Area		Problem Area
CS	Concrete and Steel Structures	EM	Electrical and Mechanical
GT	Geotechnical	EI	Environmental Impacts
HY	Hydraulics	OM	Operations Management
CO	Coastal		

For example, Technical Report REMR-CS-1 is the first report published under the Concrete and Steel Structures problem area.

COVER PHOTOS:

TOP — Deteriorated concrete gate pier, Montgomery Dam

BOTTOM — Pressure injection of concrete cracks, Bloomington Dam Intake

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Presented are the results of a program conducted to identify the repair methods and materials currently being used to repair and rehabilitate concrete structures deteriorating as a result of cracking and spalling. These repair methods and materials were evaluated for their applicability to the in situ repair of concrete hydraulic structures. From this evaluation, the following three repair techniques were identified as best suited for in situ repair procedures: pressure injection, polymer impregnation, and addition of reinforcement. Case histories illustrating the application of each technique are presented. Recommendations are made for work to be performed to make these systems more applicable to the repair of hydraulic structures.

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PREFACE

The study reported herein was authorized by Headquarters, U.S. Army Corps of Engineers (HQUSACE), under Civil Works Research Work Unit 32308, "In Situ Repair of Deteriorated Concrete," for which Mr. James E. McDonald is principal investigator. This work unit is part of the Concrete and Steel Structures Problem area of the Repair, Evaluation, Maintenance, and Rehabilitation (REMR) Research Program. The Overview Committee of HQUSACE for the REMR Research Program consists of Mr. James E. Crews, Mr. Bruce L. McCartney, and Dr. Tony C. Liu. Technical Monitor for this study was Dr. Liu.

This study was monitored by the U.S. Army Engineer Waterways Experiment Station (WES) and conducted by the Brookhaven National Laboratory (BNL) under the auspices of the U.S. Department of Energy under Support Agreement No. WESSC-85-02. This report was prepared by Messrs. R. P. Webster and L. E. Kukacka, Process Sciences Division, BNL. The study was performed under the general supervision of Mr. Bryant Mather, Chief, Structures Laboratory (SL), and Mr. John M. Scanlon, Chief, Concrete Technology Division (CTD), SL, and under the direct supervision of Mr. McDonald, CTD. Program Manager for REMR is Mr. William F. McCleese, CTD.

COL Dwayne G. Lee, CE, is Commander and Director of WES. Dr. Robert W. Whalin is Technical Director.

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CONVERSION FACTORS, NON-SI TO SI (METRIC) UNITS OF MEASUREMENT

Non-SI units of measurement used in this report can be converted to SI (metric) units as follows:

Multiply	By	To Obtain
cubic yards	0.7645549	cubic metres
Fahrenheit degrees	5/9	Celsius degrees or kelvins*
feet	0.3048	metres
gallons (U.S. liquid)	3.785412	cubic metres
gallons (U.S. liquid) per square foot	40.745836	liters per square metre
inches	25.4	millimetres
miles (U.S. statute)	1.609347	kilometres
pounds (force) per square inch	0.006894757	megapascals
square feet	0.09290304	square metres
square miles	2.589998	square kilometres

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* To obtain Celsius (C) temperature readings from Fahrenheit (F) readings, use the following formula: C = (5/9)(F - 32). To obtain kelvin (K) readings, use: K = (5/9)(F - 32) + 273.15.

IN SITU REPAIR OF DETERIORATED CONCRETE IN HYDRAULIC STRUCTURES: FEASIBILITY STUDY

PART I: INTRODUCTION

Background

Over the last 75 to 80 years, reinforced portland-cement concrete has been used extensively in hydraulic structures, such as dams, spillways, lock chambers, and bridge support columns and piers. The Corps of Engineers estimates that it now operates and maintains 536 dams and 260 lock chambers at 596 sites (Scanlon 1983). Of these, more than 40% are over 30 years old and 29% were constructed before 1940. It is further estimated that nearly half the 260 lock chambers will reach the end of their 50-year design lives by the turn of the century. Over this same period of time, waterborne traffic is expected to increase approximately 50%. With the relatively limited new construction starts anticipated, many of these structures will be kept in service well beyond their original design lives. Periodic inspections of these facilities reveal that a large number of the older structures require significant maintenance, repair, and rehabilitation. In addition, the relatively new structures must be maintained to ensure their continued service and operation.

In most cases, repairs to such structures entail removal of the deteriorated concrete and replacement with new concrete. If deterioration is not too severe, the time and costs of rehabilitation are generally acceptable. If, however, the deterioration is extensive, it is often necessary to remove substantial amounts of concrete and, in some cases, to completely remove and replace the concrete section. Considerable savings in time and cost for the rehabilitation of highly deteriorated concrete structures would be realized if methods and materials could be identified and developed to repair such structures without the extensive removal of the deteriorated concrete. To this end, Brookhaven National Laboratory (BNL) has carried out a program entitled "In Situ Repair of Deteriorated Concrete in Hydraulic Structures."

Program Objectives

The specific objectives of the BNL program were to evaluate existing methods and materials for use in the *in situ* repair of deteriorated concrete hydraulic structures, as well as to identify new materials and develop new concepts.

PART II: EVALUATION OF REPAIR METHODS

Background

Information pertaining to methods and materials used to repair portland-cement concrete hydraulic structures was obtained (a) from a computerized literature survey, and (b) from mail and telephone inquiries and meetings with government agencies and private firms active in the maintenance and restoration of concrete structures.

The objectives of the survey were twofold: (a) to identify the forms of deterioration most prevalent in concrete hydraulic structures, and (b) to identify existing methods and materials commonly used for the repair and rehabilitation of concrete structures. Once this information was collected, it was evaluated to determine the applicability of the various systems to the *in situ* repair of concrete hydraulic structures. The results of this survey are presented below.

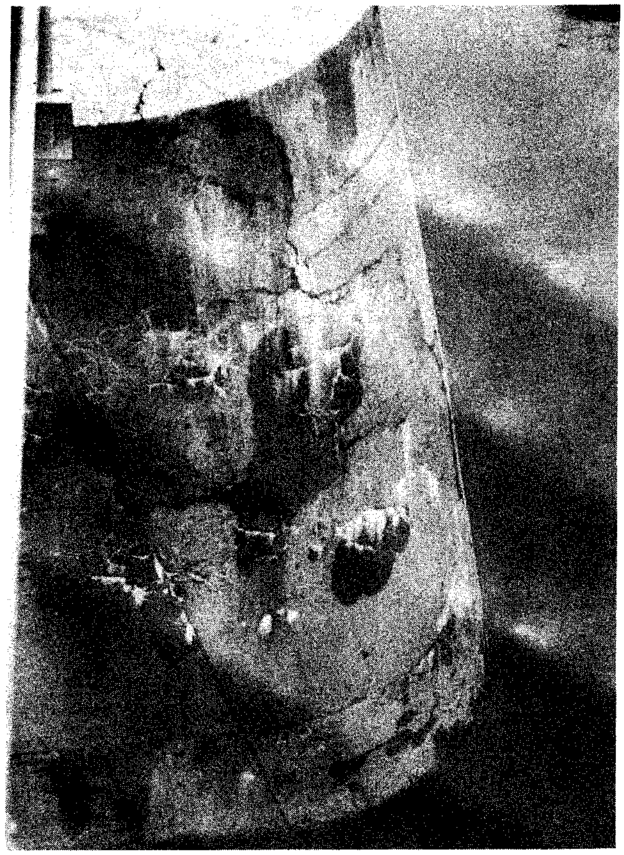
Repair and Rehabilitation Needs

In 1982, the Corps of Engineers initiated a program to develop quantitative information on the condition of the concrete portions of the Corps' civil works structures in an attempt to identify the most critical long-term needs with respect to repair and rehabilitation. This program was done, in part, by reviewing existing periodic inspection reports for Corps locks and dams in order to obtain input data for a computerized data base to be used to identify and evaluate trends in deterioration and other problem areas in concrete hydraulic structures. The results of this study (McDonald and Campbell, 1985) indicated that the most common types of concrete deficiences were (a) cracking, (b) seepage, and (c) spalling. These three general catagories of deficiencies accounted for 77% of the total of 10,096 deficiencies identified during the review of available inspection reports. Concrete cracking was the deficiency most often observed, accounting for 38% of the total. *In situ* repair procedures may not be readily applicable in the repair of seepage deficiencies. However, deterioration due to cracking and spalling does seem to be suited to the use of *in situ* repair procedures.

Cracking and spalling of concrete can have a number of causes including temperature stresses, adverse chemical reactions, corrosion of reinforcing steel, weathering (cycles of freezing and thawing), accidental overloading, and differential movement of the structure. Much of the damage typically found in many of the structures operated by the Corps apparently is due to deterioration resulting from the penetration of water into exposed surfaces which are horizontal or which have inadequate drainage followed by freezing and thawing (Figures 1-3). Vertical surfaces made using non-air entrained concrete are equally susceptible to this type of deterioration. The initial cracking in these structures may actually have any of several different causes including drying shrinkage, thermal stresses, alkali-aggregate reaction, and stresses due to structural movement; however, once the cracking has occurred, water ponding on the surfaces of the structure results in the penetration of moisture into the concrete mass. Subsequent cycles of freezing and thawing of the saturated concrete leads to progressive deterioration of the concrete, aggravated, in some cases, by the fact that non-air-entrained concrete was used in many of the older structures.

Although initial deterioration due to cracking and spalling may visually appear to be very severe, the concrete member generally will remain intact until the deterioration becomes excessive, at which time the concrete will begin to break apart. Once this stage is reached, it is generally necessary to remove and replace the deteriorated concrete. However, if the concrete can be repaired using _in situ_ repair procedures before it has deteriorated this far, it is possible to extend the service life of the structure.

With this objective in mind, the emphasis of the program was directed toward identifying repair methods and materials to be used for the _in situ_ repair of hydraulic concrete structures exhibiting deterioration due to cracking and spalling.



a. General view of deterioration due to cracking.

b. Closeup view of top of gate pier.

Figure 1. Typical cracking deterioration in concrete gate piers.

Figure 2. Typical cracking in concrete arch.

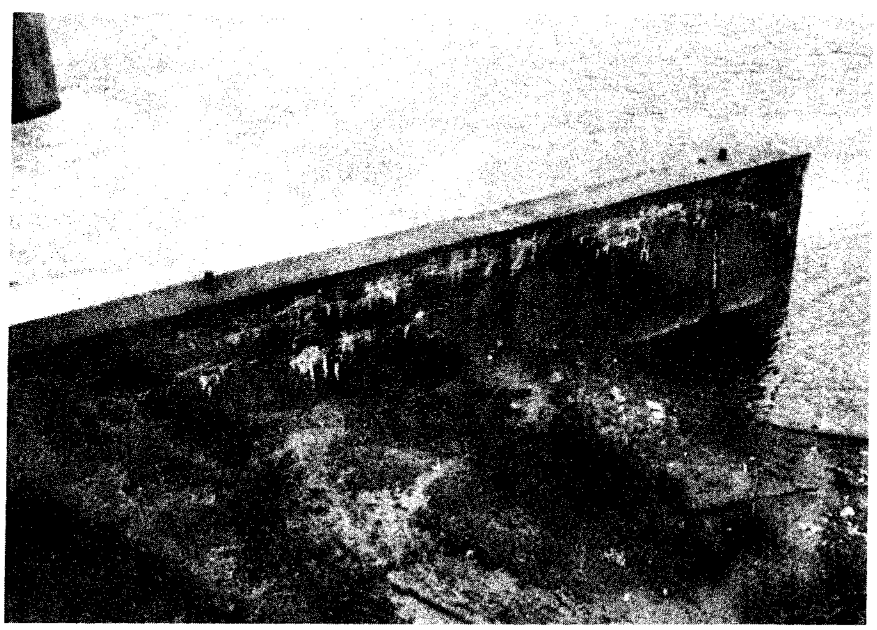


Figure 3. Typical cracking in concrete wall.

Methods and Materials for the Repair of Cracks in Concrete

Despite the general belief that it is a very durable construction material, concrete is very susceptible to deterioration due to cracking. As a result, cracking is the maintenance and repair problem most frequently encountered in concrete structures. As previously mentioned, causes for cracking include errors in design and detailing, temperature stresses, chemical reactions, corrosion of embedded metals (reinforcing steel), weathering (freezing and thawing), accidental overloadings and differential movement of the structure. The large number of factors that can cause concrete to crack indicates that no one repair procedure is appropriate in all instances. To ensure success, the cause of the cracking as well as the present condition of the crack must be taken into account in selecting a repair procedure.

In 1984, Committee 224 of the American Concrete Institute released a report detailing the causes, evaluation, and repair of cracks in concrete structures, and identified 12 techniques most commonly used for the repair of cracks in concrete (see Table 1). A discussion of each technique is presented below.

1. Pressure Injection. This repair technique, which has been used successfully to repair cracks varying in width from 0.002 to 0.003 in. up to 0.25 in., consists of drilling holes at close intervals along the length of the cracks, installing injection ports, sealing the surface of the crack, and injecting an adhesive, usually epoxy, under pressure into the crack. Injection progresses from hole to hole, normally beginning at the lowest point, and continues until the entire length of the crack has been filled. Injection has been used successfully to repair cracks in bridges, buildings, dams, and other types of structures. However, unless the crack is dormant, it will probably recur, usually elsewhere in the structure. If the crack is active and it is desirable to seal the crack while allowing continued movement at that location, it is necessary to use a sealant or other material that allows the crack to function as a joint. Injection can be used, within limits, against a hydraulic head, provided the injection pressure is adjusted upward to counteract the hydraulic head.

Injection techniques require an average degree of skill for satisfactory execution, and application of the technique is limited by ambient temperature. If the part of the structure to be repaired is subject to varying seasonal temperature changes, the width of the cracks can vary considerably. If possible, repairs should be scheduled during the cooler seasons, especially in the spring, when cracks are at their widest. Cracks filled during cooler weather will be in compression, whereas cracks filled during summer or early fall, will likely be in tension when the structure cools during winter months.

2. Routing and Sealing. Routing and sealing is the simplest and most common technique used to repair cracks that are dormant and have no structural significance. It is not applicable for sealing cracks subjected to high hydrostatic pressures, except when the pressurized face is being sealed, in which case some reduction in flow can be obtained, nor is it applicable in instances where aesthetics are important since the technique "highlights" the crack being repaired.

The technique simply entails enlarging the crack along its exposed face and filling and sealing it with a suitable joint sealant (Figure 4). Choice of the sealant depends on how tight or permanent the seal must be. Epoxy compounds are most commonly used, although hot-poured joint sealants have been used when watertightness is not required and appearance of the repair is unimportant. Urethanes, which remain flexible through large temperature variations, have been used successfully in cracks up to 0.75 in. in width.

3. Stitching. This technique involves drilling holes on both sides of the crack and grouting in U-shaped reinforcing bars (stitching dogs) that span the crack (Figure 5). A non-shrink grout or an epoxy resin is normally used to anchor the legs of the bars. Stitching is generally used when it is necessary to reestablish tensile strength across major cracks. Stitching does tend to stiffen the area being repaired, which may accentuate the overall stiffness of the structure and cause cracks to develop elsewhere. It is, therefore, generally necessary to strengthen the adjacent sections using external reinforcement.

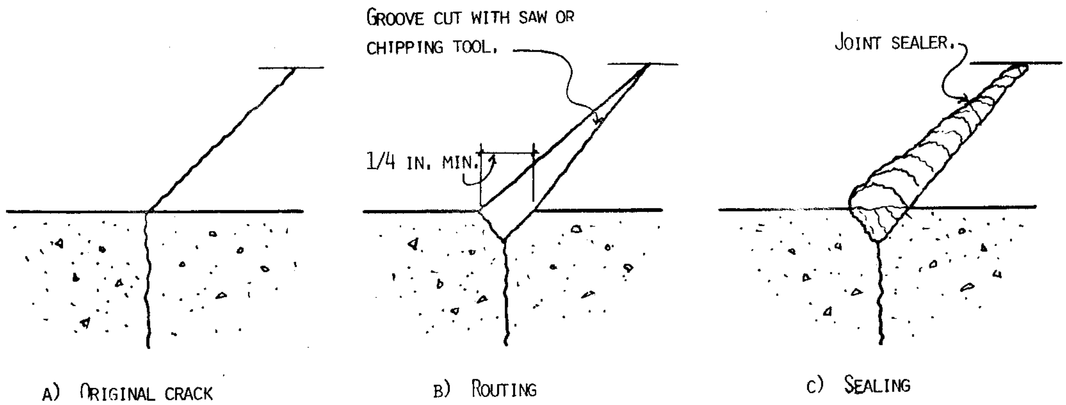


Figure 4. Repair of crack by routing and sealing (ACI Committee 224, 1984).

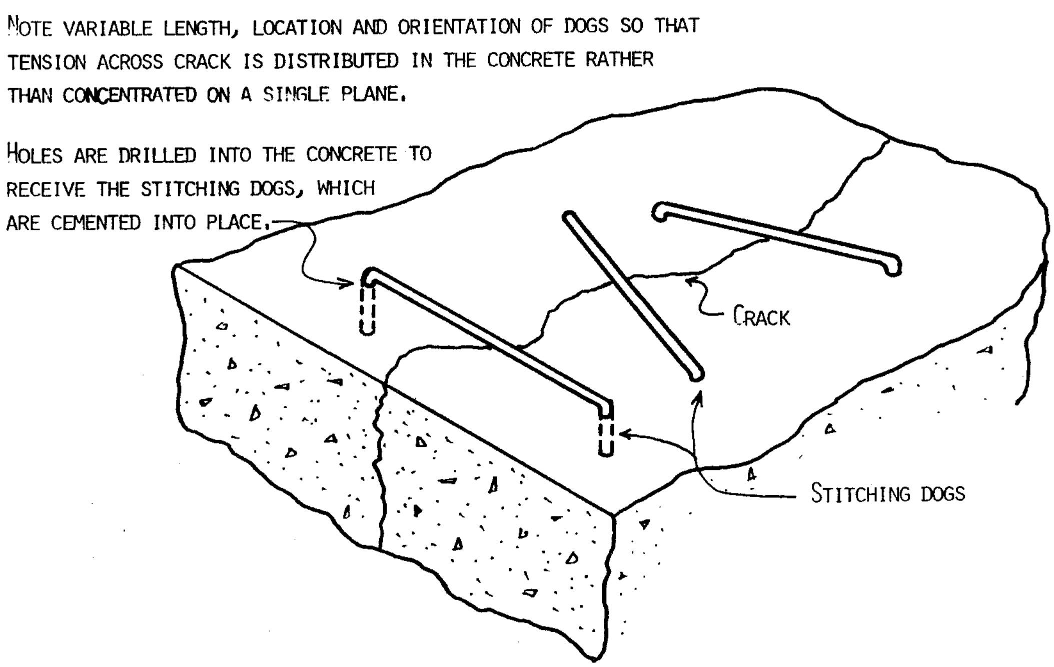


Figure 5. Repair of crack by stitching (ACI Committee 224, 1984).

Stitching will not close a crack but can be used to prevent it from propagating further. If water is a problem, the crack should be made water tight before it is stitched to protect the bars from corrosion.

4. Additional Reinforcement. Cracked reinforced concrete members have been successfully repaired by adding reinforcement to the member, both internally (post-reinforcement) and externally.

Post-reinforcement is a technique developed by the Kansas Department of Transportation to repair cracked structural bridge concrete. The method consists of sealing the surface of the crack, drilling holes at 45° to the deck surface, and crossing the crack plane at approximately 90°, filling the hole and crack plane with epoxy pumped under low pressure, and placing a reinforcing bar into the drilled hole in a position to span the crack (Figure 6). The epoxy bonds the bar to the walls of the hole; it fills the crack plane, thereby rebonding the cracked concrete surfaces in one monolithic form and reinforcing the section.

Externally applied post-tensioning is often used when a major portion of a member must be strengthened or when cracks need to be closed. The technique uses prestressing tendons or bars to apply a compressive force to the face of the member being repaired. The effects of the tensioning forces on the rest of the structure must be analyzed to prevent cracks from developing in other parts of the structure.

5. Drilling and Grouting. This technique is useful for repairing vertical cracks which are reasonably straight and are accessible at one end of the crack such as in retaining walls. It consists of drilling a hole, usually 2 to 3 in. in diameter, down the length of the crack and grouting it in to form a key (Figure 7). The grout key prevents transverse movement of the concrete adjacent to the crack and will also reduce heavy leakage through the crack. If watertightness is essential and structural load transfer is not, the drilled hole is filled with a resilient material in place of the grout.

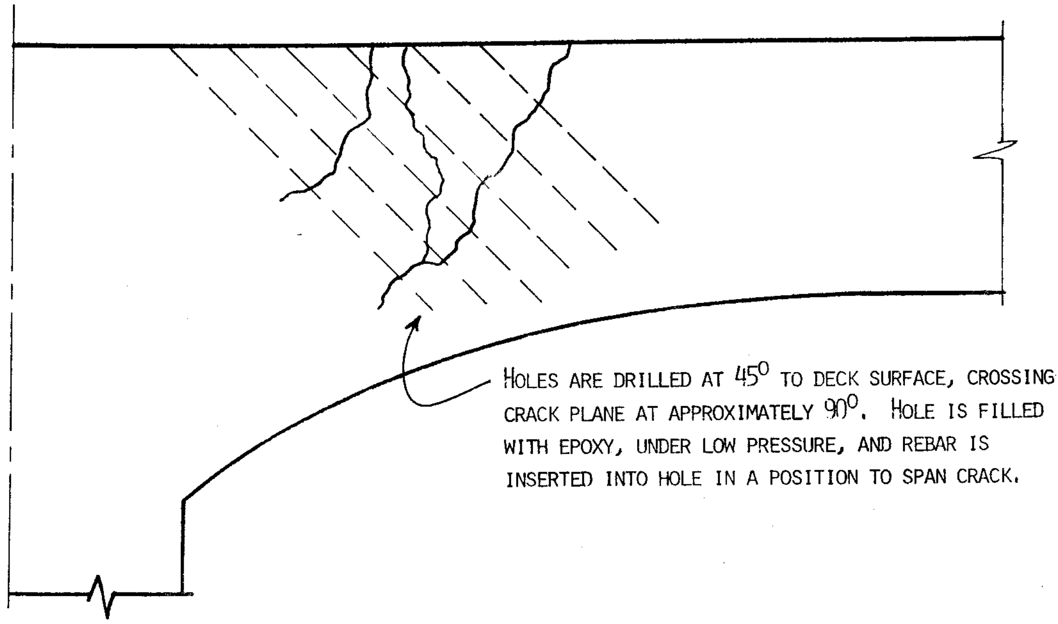


Figure 6. Repair of crack by post reinforcement.

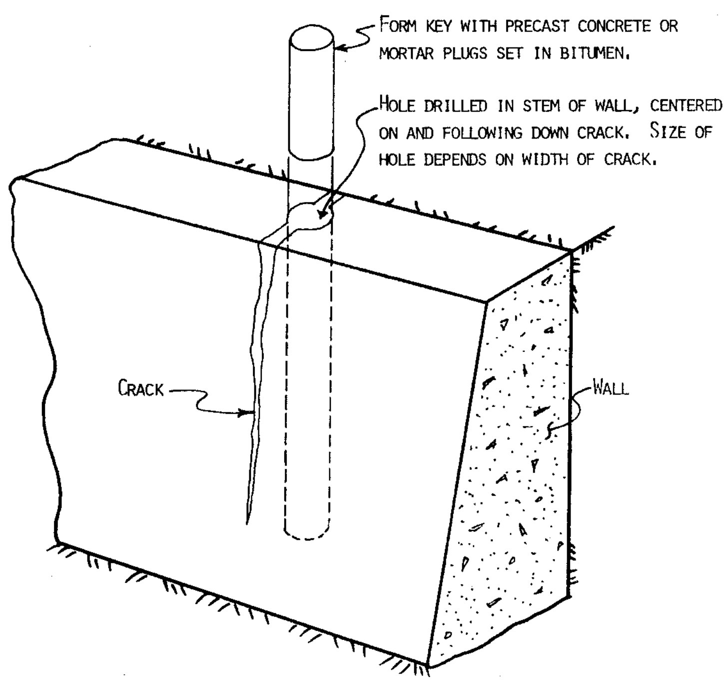


Figure 7. Repair of crack by drilling and plugging (ACI Committee 224, 1984).

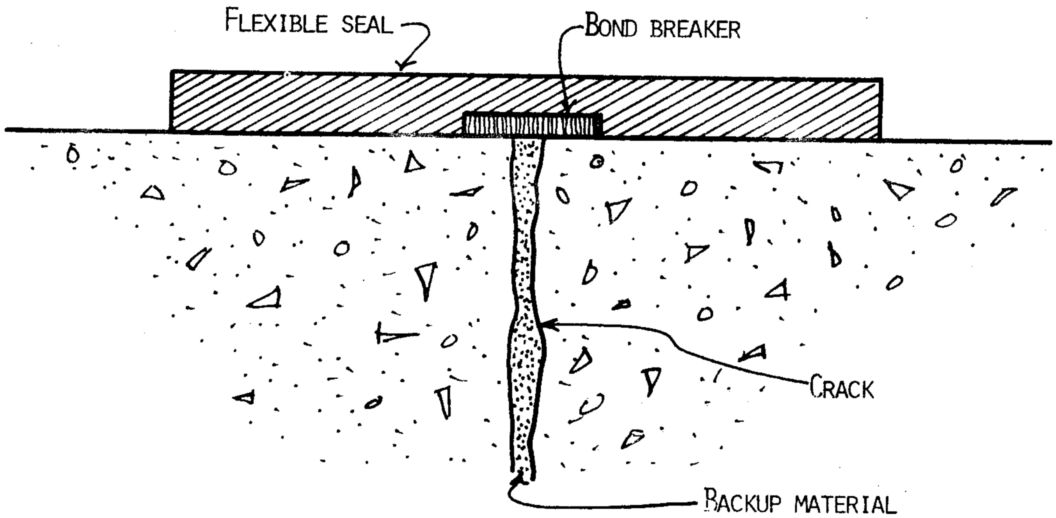


Figure 8. Repair of crack using a flexible seal (ACI Committee 224, 1984).

6. Flexible Sealing. Active cracks can be sealed by routing them out and filling them with a suitable flexible sealant. This technique is applicable where appearance is unimportant and in areas where the cracks are not subject to traffic or mechanical abuse. The repairs are made by routing out a slot or groove along the length of the crack of sufficient width and shape to accommodate the expected movement. The slot is then filled with a backing material, covered with a bond breaker, and finally sealed with the flexible joint sealant (Figure 8).

7. Grouting. Wide cracks, particularly in gravity dams and thick concrete walls, can be repaired with portland-cement grout. Narrow cracks may be repaired with chemical grouts consisting of solutions of chemicals that combine to form a gel, a solid precipitate, or a foam. Cracks as narrow as 0.002 in. have been repaired using chemical grouts. The procedure, in general, consists of cleaning the concrete along the length of the crack; installing grout nipples at intervals, and sealing the crack between the nipples; flushing the crack to clean it and to test the seal; and then pumping the grout. After the crack is filled, the pump pressure is maintained for several minutes to ensure good penetration of the grout.

8. <u>Drypack Mortar</u>. This technique consists of the hand placement of a low water content mortar followed by tamping or ramming of the mortar into place to produce tight contact between the mortar and the existing concrete. The repair exhibits very little shrinkage and the patch remains tight, with good durability, strength, and watertightness. It is appropriate for use in cavities that are deeper than they are wider. Since formwork is not required, drypack is especially appropriate for use in vertical members. It is not appropriate for the repair of extensive, wide or shallow areas or for applications that require compaction of the mortar behind obstacles, such as reinforcing bars. The technique is used for the repair of dormant cracks but is not recommended for active cracks.

9. <u>Crack Arrest</u>. This technique is commonly used to prevent reflective cracking into new concrete which is being placed over existing concrete. The technique, in general, arrests the cracking by blocking it and spreading the tensile stresses causing the cracking over a larger area. One technique consists of placing semicircular sections of steel pipe over the crack and then covering the pipe with concrete to anchor it in place. After this concrete cap has set, the new concrete is placed over the cap. The steel pipe is later filled with grout to complete the structural continuity of the member (Figure 9).

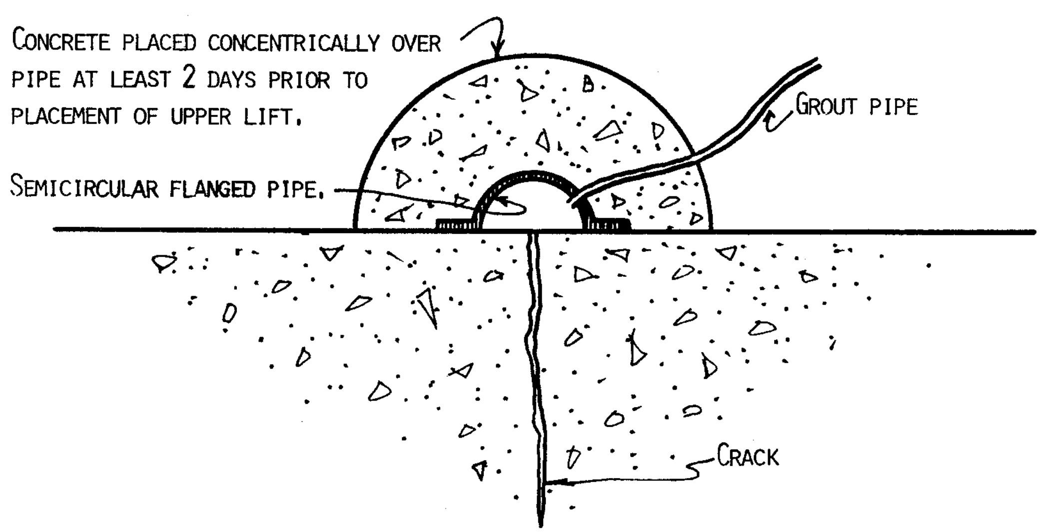


Figure 9. Repair of crack using the crack arrest method (ACI Committee 224, 1984).

10. <u>Polymer Impregnation</u>. Surface impregnation techniques have been used as a means for reducing chloride and moisture penetration into concrete bridge decks by filling the pore structure in the concrete with polymer. However, it has also been successfully used to restore the structural integrity of highly deteriorated concrete bridge decks and floor slabs as well as to improve the abrasion resistance of the concrete in outlet tunnel walls of dams. The impregnation process consists of four basic steps:

a) preparation of the surface to remove contaminants or films that would prevent or reduce monomer penetration,

b) drying the concrete to a depth sufficient to permit the desired monomer penetration,

c) impregnation of the concrete with liquid monomer to the desired depth,

d) polymerization of the monomer within the pores of the concrete.

Vacuum impregnation techniques are similar to surface impregnation except that the liquid monomer is forced into the pore structure under pressure, in this case a negative pressure, to produce greater penetration of monomer into the concrete. It also provides a method for impregnating surfaces which are not compatible with the gravity-soak method of impregnation such as overhead and vertical surfaces.

11. <u>Overlays and Surface Treatments</u>. Concrete containing fine dormant cracks can be repaired by applying a bonded overlay to the surface. However, most cracks are subject to movement caused by variations in loading, temperature, and moisture. Cracks subjected to such forces will reflect through any bonded overlay, defeating the purpose of the overlay insofar as repair of cracks is concerned. Bonded overlays are also subject to cracking due to drying shrinkage and restraints at the interface. Unbonded overlays can be used to cover surfaces with moving cracks. However, thin unbonded overlays which receive load may crack due to a combination of thermal and load stresses.

Cracks may also be repaired by sealing the surface of the member with a sealant such as an epoxy resin. However, concrete on grade in freezing climates should never receive a surface treatment which will act as a vapor barrier. This would allow moisture passing from the subgrade or surrounding environment to condense under the barrier, leading to critical saturation of the concrete and rapid deterioration by freezing and thawing.

12. Autogenous Healing. Autogenous healing is a natural process of crack repair that can occur in concrete in the presence of moisture and the absence of tensile stresses. This healing process is dependent upon the carbonation of calcium hydroxide in the cement paste by carbon dioxide present in the surrounding air and water. Calcium carbonate and calcium hydroxide crystals form and grow within the cracks. The chemical and mechanical bonding resulting between the crystals and the surfaces of the paste and the aggregate restores some of the tensile strength of the concrete across the cracked section, and the crack may become sealed. Healing will not occur if the crack is subject to movement or if there is a positive flow of water through the crack.

Methods and Materials for Repairing Spalled Concrete

As with the repair of cracks in concrete, a number of methods and materials have been used to repair concrete subjected to surface spalling and scaling. This section covers the basic repair techniques and materials most commonly used (Concrete Construction, 1982). The techniques and materials are presented in Tables 2 and 3 and are discussed below.

Repair Methods

1. Coatings. This technique is generally used when scaling or spalling is limited to a very thin region at the surface of the concrete. It consists of painting a film-forming plastic or liquid coating over the surface of the concrete. Coatings can be applied by brushing, rolling, or spraying. Some common applications for coatings are reduction of ingress of water, protecting concrete from aggressive chemicals, and providing a durable wearing surface under heavy traffic loads.

2. Concrete Replacement. Unsound concrete can be removed and replaced with conventional portland-cement concrete or some other patching material. Whether the existing concrete is completely or only partially removed depends upon the extent and nature of the deterioration. This technique is one of the most commonly used and is appropriate for applications where the cause of the deterioration is nonrepeating or has been eliminated.

3. Grinding. This technique can be used when the deterioration is limited to a thin region at the surface of the concrete. However, unless modern heavy-duty equipment is used, it can be a slow, expensive, dusty method of repair.

4. Jacketing. This technique entails fastening a material to the existing concrete that is more resistant to the environment that is causing the deterioration. The material can be metal, rubber, plastic, or high-strength concrete; it can be secured to existing concrete by bolts, nails, screws, adhesives, straps, or gravity. Common applications include tanks, spillways, piers, and other concrete elements that are exposed to corrosive materials or rapidly flowing water.

5. Shotcreting. This technique entails shooting concrete or mortar under pressure into the cavity or onto the surface of the concrete to be repaired. It may involve pumping completely mixed material through the hose or blowing the dry constituents through the hose and mixing them with water at the nozzle. The latter method requires an experienced operator but offers the capability of customizing the shotcrete water content and consistency to the needs of specific areas of the repair job. Shotcrete is practical for large jobs, on either vertical or horizontal surfaces, where the cavities are relatively shallow.

6. Preplaced-Aggregate Concrete. This technique which is also called "prepacked" concrete, entails prepacking gap-graded aggregate into the cavity to be repaired and inundating the cavity with water to saturate the aggregate. Mortar or grout is then pumped in from the bottom,

displacing the water. This technique is suitable for inaccessible applications, such as submerged concrete or deteriorated concrete that is being jacketed. Preplaced-aggregate concrete has been used to repair piles, footings, piers, retaining walls, abutments, base plates, tunnels, and dams. Preplaced-aggregate concrete provides low shrinkage and good bonding qualities but can leave voids. Because of the specialized skills and equipment required, this type of repair work is usually performed by a firm that specializes in it.

7. Thin-Bonded and Unbonded Overlays. Thin overlays, i.e., overlays of 2 in. or less, are often used to repair concrete surfaces that are basically sound structurally but have deteriorated from cycles of freezing and thawing, heavy traffic, or other exposures which the original concrete was unable to withstand. Overlays are also occasionally used to relevel slabs or to reestablish grades.

Once a decision has been made to place an overlay, it must be determined whether the overlay should be bonded or unbonded. If the deterioration to be repaired is a surface phenomenon, such as spalling or scaling, the overlay is usually bonded. In the case of cracking or structural movement, however, it may be desirable not to bond the overlay so that it will not reflect the distress in the base slab.

Repair Materials

1. Bituminous Coatings. Asphalt- or coal tar-based bituminous coatings are used to reduce the tendency of water to enter concrete or to protect it to some extent from weathering. They are low in cost, familiar to workmen, and effective for a time, if properly applied; and their thickness, and therefore, their resistance to water passage and weathering, can be varied to suit the exposure. The disadvantages of bituminous coatings include the need to replace or renew them periodically, the messiness and the odors associated with their application, their tendency to dry and crack, their sensitivity to ambient temperatures, and the rapid destruction of the coatings when certain liquids, such as gasoline, are spilled on them.

2. Concrete, Mortar, and Grout. Portland-cement concrete, mortar, and grout have a number of advantages as repair materials, including thermal properties similar to those of the existing concrete, similarity in appearance, relatively low cost, easy availability, and familiarity. Concrete is most often used for the complete replacement of sections and deep cavities extending beyond the top layer of reinforcing steel. Mortar is generally used for cavities of 1 1/2 in. or less, or for applications that are too shallow for the coarse aggregate in concrete and where the fluidity of grout is unnecessary or not wanted. Grout has the advantages of being fluid and readily pumped even into areas that cannot be seen. Grout can be used where clearances are minimal and where it is necessary to reduce the likelihood of leaving major voids. Grouts, on the other hand, have a higher water content and consequently undergo more drying shrinkage than well-designed mortar or concrete.

3. Epoxies. Epoxies are acknowledged to be excellent repair materials for selected applications. Epoxies are organic compounds that, when mixed with a hardening agent, produce a tough, rapid-setting, and rapid-hardening material that is chemically and physically stable, and that is resistant to water penetration, crack formation, and many chemicals that attack concrete. They also have excellent adhesive properties, and they may be modified. For example, they can be made rubbery for elastomeric sealing or caulking applications, or fine hard aggregate can be broadcast over the top of a coating to produce a skid-resistant surface. Epoxies have the following disadvantages: high cost, allergenic effects on some workmen, and significant differences from concrete in important physical properties such as coefficient of thermal expansion, tensile strength, and flexural strength. Epoxies are most often used in concrete repair work as an adhesive to bond new concrete to hardened concrete, for patching, and for coatings.

4. Expanding Mortars, Grouts, and Concretes. These materials are generally proprietary materials that counteract the problem of shrinkage by incorporating ingredients which produce an expansion approximately equal in magnitude to the expected drying shrinkage. Variations in the effectiveness of these materials are such that performance data and

previous applications should be checked before use. Some materials have been found to produce excellent long-lasting repairs while others have exhibited lack of density and other deficiencies that could undermine their effectiveness.

5. Linseed Oil. Linseed oil has been used to correct scaling which is not severe enough to warrant using a coating or some other form of repair. Strictly speaking, linseed oil is not a repair material because it is used to prevent or minimize additional scaling, not to repair existing scaling. The linseed-oil solution is applied to the slab surface and penetrates up to 1/8 in., providing a film of low permeability that resists infiltration by aggressive solutions while allowing water vapor to escape.

6. Latex-Modified Concrete. Concrete and mortar that have been modified through the addition of a latex admixture have been used successfully to repair deteriorated floors and bridge decks. Latex-modified concretes typically exhibit good bond strength to existing sound concrete as well as high compressive and tensile strengths, and they are more flexible than unmodified mixtures and are resistant to weathering, alkalies, and dilute acids. Though more expensive than unmodified portland-cement concrete, latex-modified concrete is low in cost compared with many other types of repair materials, including those based on epoxies. On the other hand, they are in some ways more difficult to handle and finish than other materials since they set rapidly and form a skin that tears, if troweled after the skin has formed.

7. Polymer Concrete Materials. Polymer concrete patching materials have been used extensively in the repair of highway bridge decks and pavements. Unlike normal portland-cement concrete, polymer concrete contains no water and, generally, no portland cement. Instead, the blended aggregate filler is bonded together in a polymer matrix. A number of materials such as methyl methacrylate, unsaturated polyesters, vinyl esters, polyurethane, and epoxy have been used as the polymer matrix. Polymer concrete materials have a number of advantages over normal portland-cement concrete including rapid curing characteristics, high early strength, good bond strength, impermeability to water penetration, and excellent resistance to freezing and thawing . Disadvantages include the

need to pay detailed attention to manufacturers' recommendations, careful formulation necessary to avoid problems due to differences in the physical properties of the polymer concrete and the adjacent portland cement-concrete, the relatively high cost and the flammability and toxicity of some of the constituents.

Selection of Recommended *In Situ* Repair Techniques

From an evaluation of the repair techniques and materials summarized above, five procedures--three crack repair techniques and two techniques for repairing spalled concrete--have been identified as being the most applicable for use as *in situ* repair procedures for concrete hydraulic structures. The selected repair techniques include pressure injection, polymer impregnation, and additional reinforcement. In conjunction with these repair procedures, thin reinforced overlays and shotcrete can be used to repair spalled concrete surfaces as well as to resurface structures after the cracks have been repaired. Criteria considered in the selection of the repair techniques identified above included the following:

The repair technique should (a) effectively repair both dormant and active cracks, (b) restore the structural properties of the cracked member, (c) provide a watertight repair of the cracks, (d) allow for the repair of the member in the presence of moisture or hydraulic pressures, (e) improve the durability of the concrete member, (f) prevent access of corrosive agents into the concrete, and (g) leave no visible scars or surface blemishes on the repaired member.

Since no single method of repair would satisfy each of the above criteria, it was necessary to identify more than one method of repair. Presented below is a discussion of each of the selected techniques.

1. Pressure Injection. Pressure injection, generally using epoxy adhesives, has been used extensively for about 25 years to repair a variety of concrete hydraulic structures including dams, spillway tunnels, bridge piers, and water-retaining tanks. As a result, much of the expertise, technology, materials, and equipment necessary for the successful application of the process already exists.

A principal advantage of the pressure injection technique is that it seals cracks externally and internally as opposed to such techniques as routing and sealing and flexible sealing, which only seal the cracks externally. The advantage of repairing the cracks internally as opposed to only sealing them externally is that complete sealing of the full depth of the crack will eliminate areas in which moisture may collect, thereby reducing the possibility of damage due to freezing and thawing. Internal sealing also helps to restore the structural integrity of the member being repaired. Cracks as fine as 0.002 in. have been repaired using pressure injection techniques.

Another advantage of this technique is that with proper selection of a water-compatible adhesive, cracks saturated with water can be repaired. Pressure injection can be used, within limits, against a hydraulic head, provided the injection pressure is adjusted upward to counteract the pressure of the hydraulic head.

While pressure injection is generally very successful when done properly, it does have some limitations. For example, the process may leave scars on the surface of the member where the cracks have been injected, unless the surface of the member is refinished once the repairs have been completed. The application of the process is generally limited to members which have not yet begun to spall and to the repair of dormant cracks. The process can perhaps be used for partial repair of active cracks; however, continual movement of the structure will lead to new or continued cracking.

Pressure injection, however, appears to be one of the most viable methods available for the repair of severely cracked structures, such as those shown in Figures 1 through 3.

2. Polymer Impregnation. Although research regarding the basic material properties of polymer-impregnated concrete has been in progress for about 17 years, practical *in situ* applications on existing structures have taken place only in the last 8 to 10 years. Structures repaired and

rehabilitated with polymer impregnation include highway bridge decks, structural floor slabs, outlet tunnel walls, and stilling basins. It should be pointed out that most of these applications have been experimental in nature, and routine applications of the process are yet to become commonplace.

Although the laboratory research and the field applications have shown that polymer impregnation is an excellent method for rehabilitating highly deteriorated concrete, the process has several limitations which currently prevent its use from becoming routine. The limitations include the following:

Because the process is relatively new and uses specialized materials and equipment, a relatively high level of expertise and supervision is required, and it cannot be performed by untrained or unsupervised personnel. The monomer systems currently being used to impregnate concrete require specialized safety procedures since they are often flammable and toxic. In addition, the monomers are not water compatible. Therefore, to ensure complete penetration of monomer into the pore structure of the concrete, the concrete must be dried to remove the free moisture within the pores. These limitations can make polymer impregnation a relatively expensive method of repair, when compared to more conventional methods.

Despite these limitations and restrictions, polymer impregnation appears to offer a very good method for rehabilitating or improving the overall physical and mechanical characteristics of highly deteriorated, low-quality, or non-air-entrained concrete. Properties of polymer-impregnated concrete which make it very attractive for use in concrete hydraulic structures include low permeability to water and chloride penetration, excellent long-term resistance to freezing and thawing, and compressive and flexural strengths three to four times greater than those of ordinary concrete.

3. <u>Addition of Reinforcement</u>. As previously mentioned, cracked reinforced-concrete members have been successfully repaired and upgraded by adding additional reinforcement to the member either internally or externally. The addition of internal reinforcement, referred to as post reinforcement, has been used successfully for a number of years by the Kansas Department of Transportation, which developed the process to repair cracked bridge deck beams and girders. As a result, much of the necessary materials, equipment, and expertise necessary for the successful application of the repair technique already exists. However, since apparently all applications of the technique have been limited to bridge deck beams and girders, it may be necessary to modify or redesign some of the equipment in order to use it with other types of structures.

External reinforcement, i.e., the addition of steel rods, plates, and reinforcing tendons to the exterior surface of a member, has primarily served as a means to upgrade the strength of an under-reinforced or highly cracked concrete member, although it can be used as a means to pull cracks closed. The major limitation regarding this form of repair is that strengthening and stiffening the member being repaired may lead to cracking in other parts of the structure.

Both methods, however, appear to offer one of the best means of repair when it is necessary not only to seal cracks but also to restore or upgrade the structural characteristics of a deteriorated member.

4. <u>Thin Reinforced Overlays and Shotcrete</u>. Once the cracks in a member have been repaired, it may be necessary to resurface the face of the member to cover up any scars or imperfections left by the repair procedure, to repair minor damage resulting from spalling, or to provide the member with a more durable wearing surface. Thin reinforced overlays, either bonded or unbonded, and shotcrete made using either conventional portland-cement mortar or concrete-polymer materials appear to offer the best approach. It should be pointed out that isolated patching of the surface may be necessary before renewing the surface, if damage due to cracking and spalling is excessive.

Both methods, overlaying and shotcreting, offer a very practical means for cosmetically renewing the surface of a member once it has been repaired. It may be necessary, however, to use reinforcing in both repair procedures to reduce the possibility of reflective cracking in the new surface. The use of concrete-polymer materials offers a means of providing surfaces with superior durability characteristics as well as of reducing the ability of water to penetrate into the member once it has been repaired.

The equipment and expertise required for shotcreting already exist, although some work may be necessary in these areas if concrete-polymer materials are to be shotcreted. Work in these areas may also be necessary in order to optimize the placement of overlays to vertical surfaces.

PART III: CASE HISTORIES

Presented in this section are case histories illustrating in situ applications of each of the three repair techniques recommended in the preceding section for use in the repair of cracked concrete hydraulic structures.

Pressure Injection

As mentioned earlier, pressure injection, primarily with epoxy resins, has been used extensively for about 25 years to repair various concrete hydraulic structures. Examples of hydraulic structures repaired with this technique include the Daniel Johnson Dam in Northern Quebec and the Pacoima and Twin Lake Dams in California. Pressure injection has also been tested, on a small scale, by the Rock Island District of the U.S. Army Corps of Engineers to repair concrete pier stems in Lock and Dam No. 20 on the Mississippi River, Canton, Missouri.

1. Daniel Johnson Dam. In 1981, massive cracks in the main arch buttress and on the face of the first arch to the left of the central arch were repaired using epoxy injection (Adhesives Engineering Bulletin, 1981a). The cracks were discovered during construction on a turbine addition to the structure. Located on the Manicaughan River in Northern Quebec, the Daniel Johnson Dam is a 700-ft-tall multi-arch dam with a reservoir covering 800 sq mi of water.

An inspection of the dam indicated that two major fissures needed to be repaired. The first was a shallow 4,000-ft^2 fissure intersecting the face and penetrating to a depth of approximately 5 ft, and the second was a 7,000-ft^2 crack ranging from 7 to 12 ft in depth. The shallow fissure was generally wider than 1/4 in., while the deeper crack was less than that.

After the damage was inspected and evaluated, a repair system was developed which called for the grouting of the cracks that were >1/4 in. with a cement slurry and injecting the cracks <1/4 in. with a structural epoxy adhesive.

Work began by drilling 1-in. diam holes on 5-ft centers to intercept the shallow fissure. Steel anchor rods were grouted into some of these holes while others were used as grouting ports. The surface of the fissure was sealed with a cement paste. Cement grout was then pumped into the fissure to fill all the large voids. After the grouting was completed, cores were taken, and any small fissures which were found to be empty were grouted with epoxy adhesive.

The deeper and narrower fissure was repaired in a similar manner, using the epoxy adhesive. However, dyed water was first injected into the crack to determine the communication pattern and the volume of adhesive needed to fill the crack. Upon completion of the dye test, the cracks were pressure injected with the epoxy adhesive. A total of approximately 1000 gal of Adhesive Engineering's Concresive 1380 injection adhesive epoxy was used to seal the cracks.

2. Pacoima Dam. In 1972, over 470 lin. ft of cracks were repaired in the walls of the spillway tunnel of the Pacoima Dam, located about 4 mi northeast of San Fernando, California, in the Pacoima Creek (Adhesives Engineering Bulletin, 1973).

The reinforced concrete dam, completed in 1929, has a crest height of 365 ft above the original streambed, a crest length of 640 ft and a base thickness of 100 ft. The 300 ft long spillway tunnel, cut through a mountain, has two inlets. The tunnel is 15 ft in diameter and has a 12-in. thick reinforced concrete wall.

The smaller cracks in the tunnel walls were caused by aging and the temperature extremes in the tunnel, and the larger cracks by the severe shocks of the February 1971 earthquake in the San Fernando Valley. The cracks varied in thickness from hairline to 1/4 in. Water seepage through the cracks and construction joints had discolored the concrete and created a buildup of efflorescence.

The damage was repaired by first thoroughly wire brushing the cracks and construction joints to remove all traces of dirt and loose foreign material. Where required, both temporary seals and permanent epoxy

adhesive seals were placed over the cracks. Injection ports were placed approximately 6 in. on center along the sealed cracks. Adhesive Engineering's Concresive 1050 epoxy adhesive was then injected at moderate pressure from port to port until all cracks were penetrated fully and sealed.

During the repair work, extreme weather conditions were encountered, including below-freezing ambient temperatures. Space heaters brought into the tunnel to help raise the ambient temperatures did not help very much. Nevertheless, the epoxy cured out completely, although curing took longer than usual.

Cores removed from the repaired tunnel walls revealed complete penetration of the cracked concrete, even to the finest branching cracks.

3. Twin Lakes Dam. Constructed in 1923, the Twin Lakes Dam is a reinforced concrete arch structure located on Caples Lake near Carson Pass in the California Sierras (Adhesives Engineering Bulletin, 1971). The dam is about 30 ft high and 70 ft wide. The main arch is 18 in. thick at the bottom, tapering to 10 in. at the top.

Vertical cracks at five different locations in the face of the arch section totaled approximately 120 lin. ft and ranged from hairline up to a maximum of 1/8 in. in width. Water was leaking through some of the cracks. In addition, the concrete had begun to spall around the cracks and efflorescence was present in much of the area to be repaired.

In some areas, conventional patching materials used in the past to repair 1/16-in.-wide dry cracks had disbonded and spalled off. The worst leak in the main arch developed in one large diagonal crack which had been surface grouted.

To repair the concrete arch, the reservoir was first dewatered to allow access to the spillway from both front and back sides. The concrete surfaces were then cleaned thoroughly to remove loose concrete, dirt, and efflorescence, and the cracks and adjacent concrete were washed down with a solvent.

A surface seal was applied over the cracks on the front and back faces. Steel ports were inserted at intervals along the cracks. Adhesive Engineering's Concresive 1050 epoxy adhesive was then pressure injected through the ports for the full depth of the cracks.

Because the repair work was performed in September and October, a plastic film structure was erected around each of the five areas to be repaired to help shield the work areas from the extremely cold temperatures normally encountered in the Sierras during these months. Ambient temperatures within the enclosures were 65°F to 70°F. This assured optimum conditions for rapid cure of the epoxy adhesive.

4. Lock and Dam 20, Pier 39, Canton, Missouri. One of the best documented applications of the epoxy pressure-injection repair technique found was the work done by the Rock Island District of the U.S. Army Corps of Engineers (Flock and Walleser, 1983). In August 1982, Pier 39 of the Lock and Dam 20 on the Mississippi River, Canton, Missouri, was repaired using epoxy injection. The work was primarily done to evaluate the applicability of the repair procedure.

Construction of Lock and Dam 20, using non-air entrained concrete, was completed in 1935. Periodic inspections of the structure over the years indicated extensive cracking in many of the concrete pier stems supporting the dam service bridge. Cracking was primarily attributed to structural stresses. This cracking, however, was apparently aggravated by freezing and thawing due to inadequate drainage on the pier tops which allowed rainwater to pond on the horizontal tops of the piers and infiltrate down through the piers.

As a result of the apparent continuing deterioration of the concrete, a private contractor performed a prototype test to investigate the use of epoxy injection as an alternative to conventional repair methods of removal of unsound concrete and replacement with air-entrained concrete. The proposal submitted by the contractor called for drilling 1-in.-diam ports in areas of severe damage. Water would be pumped into the ports under pressure to measure flow quantities, trace flow patterns, and estimate the severity of the damage. Cracks on the exterior of the

pier from which water flowed during the test would be surface sealed prior to injection. Following surface sealing of the cracks, a grid of 1-in.-diam injection ports would be drilled in areas of high water flow. A two-component low-viscosity epoxy adhesive, Adhesive Engineering's Concresive 1380, would be injected into the ports through packers. The proposal indicated that cracks 0.002 in. or wider would be sealed.

Before beginning repair operations, ultrasonic pulse velocity measurements were taken through the concrete in the pier. It was anticipated that the comparison of data obtained by taking sonic readings before and after epoxy injection would be useful in evaluating the effectiveness of the epoxy repair.

Results of the preliminary pressurized water test indicated the existence of a more extensive crack network than was originally anticipated. As a result, the original work proposal was modified to require the exterior surface of the pier to be completely sealed in order to prevent leakage of the epoxy at the pressures required for injection.

Before the exterior surface of the pier was sealed, lime deposits and other loose material were removed from the pier by a portable sandblast unit. As these deposits were removed, water from the preliminary pressure test continued to seep from various cracks, especially inside the archway.

Several epoxies were used to seal the exterior of the pier. Epoxies of thick consistency were applied by trowel to the upstream and downstream faces. These were the areas which exhibited the greatest amount of surface cracking. Thinner epoxies were applied to the ceiling of the archways, walls of the access manhole, exterior walls, and the top of the pier. Silica sand was broadcast into the epoxy applied to the top of the pier to provide skid resistance.

After the epoxies used to seal the pier had cured, additional pressurized water tests were performed at a maximum water pressure of 40 psi. Several small leaks were detected in the surface seal. These areas were resealed prior to epoxy injection.

Epoxy injection was started in the middle of the archway ceiling at Port No. 9 (refer to Figures 10-12). Pumping was continued, while the epoxy reached injection Ports Nos. 2, 3, and 12 on the downstream face. The operation was then moved to the upstream ceiling port (No. 8). The contractor had difficulty maintaining a seal around the injection packer. In addition, leakage of epoxy was noted at the bottom edge of the access manhole and adjacent to the electrical conduits embedded in the archway ceiling. An attempt was made to inject epoxy into Port No. 10; however, it was impossible, even at a pressure of 160 psi.

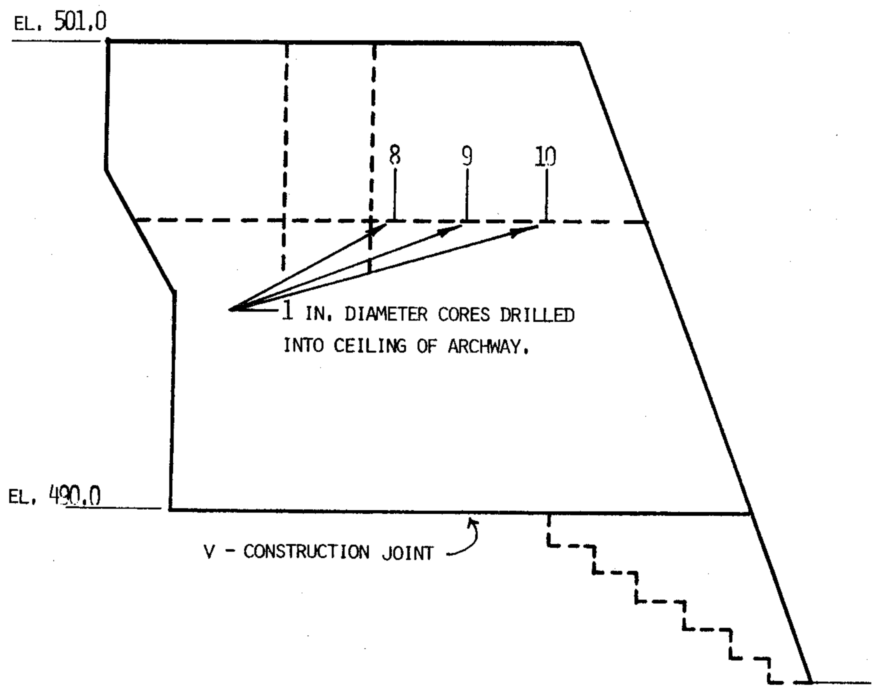


Figure 10. East elevation showing location of injection ports (Flock and Walleser, 1983).

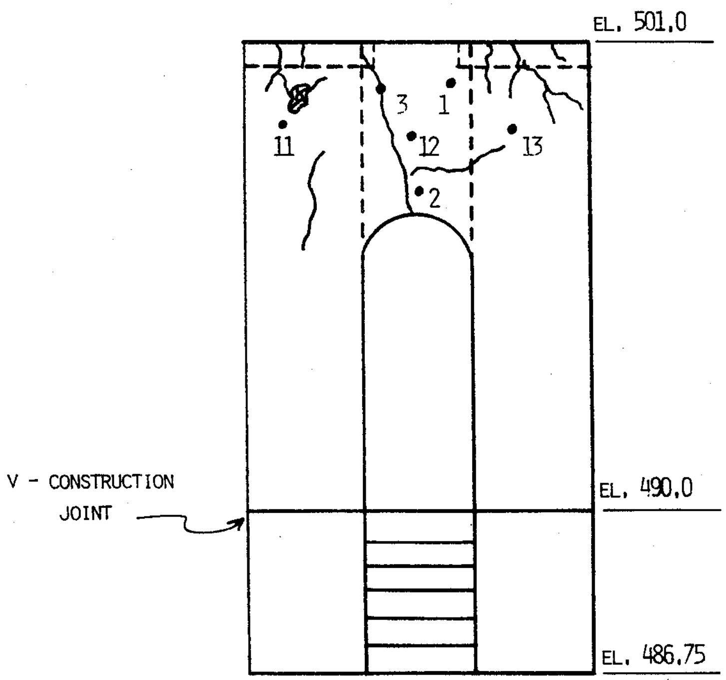


Figure 11. Downstream view showing major cracks and locations of injection ports (Flock and Walleser, 1983).

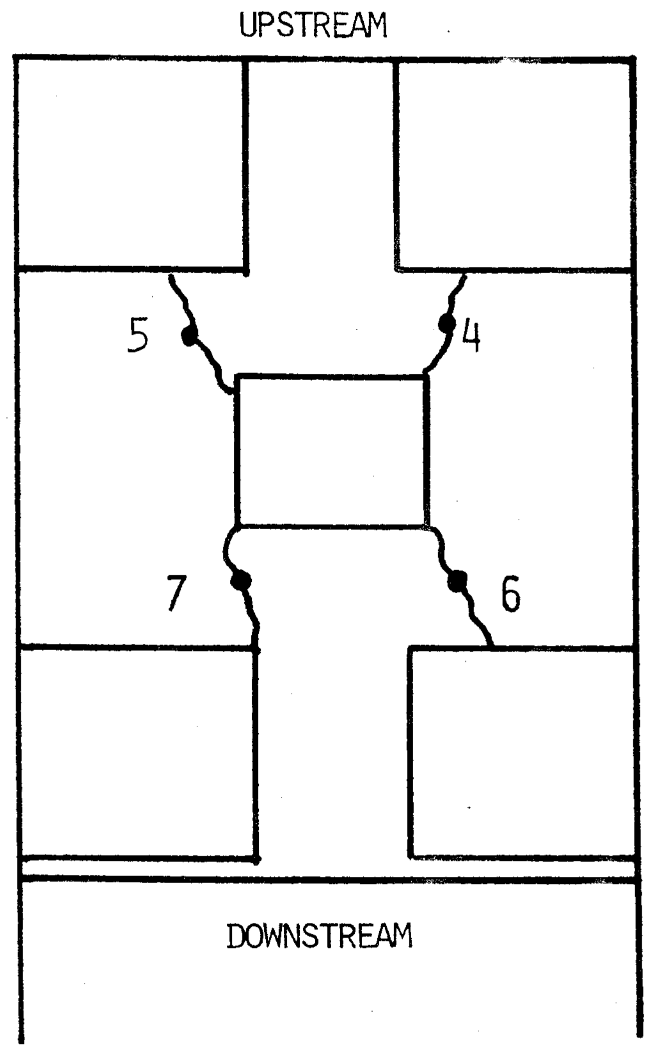


Figure 12. Top view showing major cracks and locations of injection ports (Flock and Walleser, 1983).

The injection operation was then moved to Port No. 12 on the downstream face. Again, problems were encountered with leakage at the packer seal. Injection was moved to Port No. 11. Leaks through the seal coat developed at pop-outs and along wider cracks on the downstream face. As epoxy was pumped into the pier, water was driven from the cracks and appeared on the surface of the pier at elevations below the seal coat. Epoxy also appeared at several small cracks in these areas. Injection was continued at Port No. 13, then at Port No. 1. Epoxy ponded at the top of the downstream east grout pad while pumping at Port No. 1. Injection was continued at Port No. 6 on the pier top. The last port injected was No. 7, also on top. This port was injected until epoxy appeared at the handhold at the top of the access manhole. The injection packers were then removed and all ports were sealed with epoxy. The pier was injected for a total time of approximately 34 hours. Approximately 8.4 gal of epoxy were injected into the pier.

Ultrasonic pulse velocities taken after the pier had been injected showed that some of the stations exhibited an increase in velocity over the original values. Higher velocities generally indicate sound concrete. The most significant increases occurred at locations on the lower part of the pier. Again, consistent readings could not be obtained around the top of the pier, indicating greater deterioration in the concrete in the upper portion of the pier stem.

Upon completion of the pulse velocity tests, concrete cores were taken from various locations to further evaluate the effectiveness of the repair. Presented in Figure 13 are photographs of cores taken before and after the pier was repaired. Cores removed from the pier top contained numerous random and crudely parallel diagonal cracks as well as many fractured aggregate particles. Only 5 to 10% of the cracks were filled with epoxy, explaining the difficulty in obtaining pulse velocity readings.

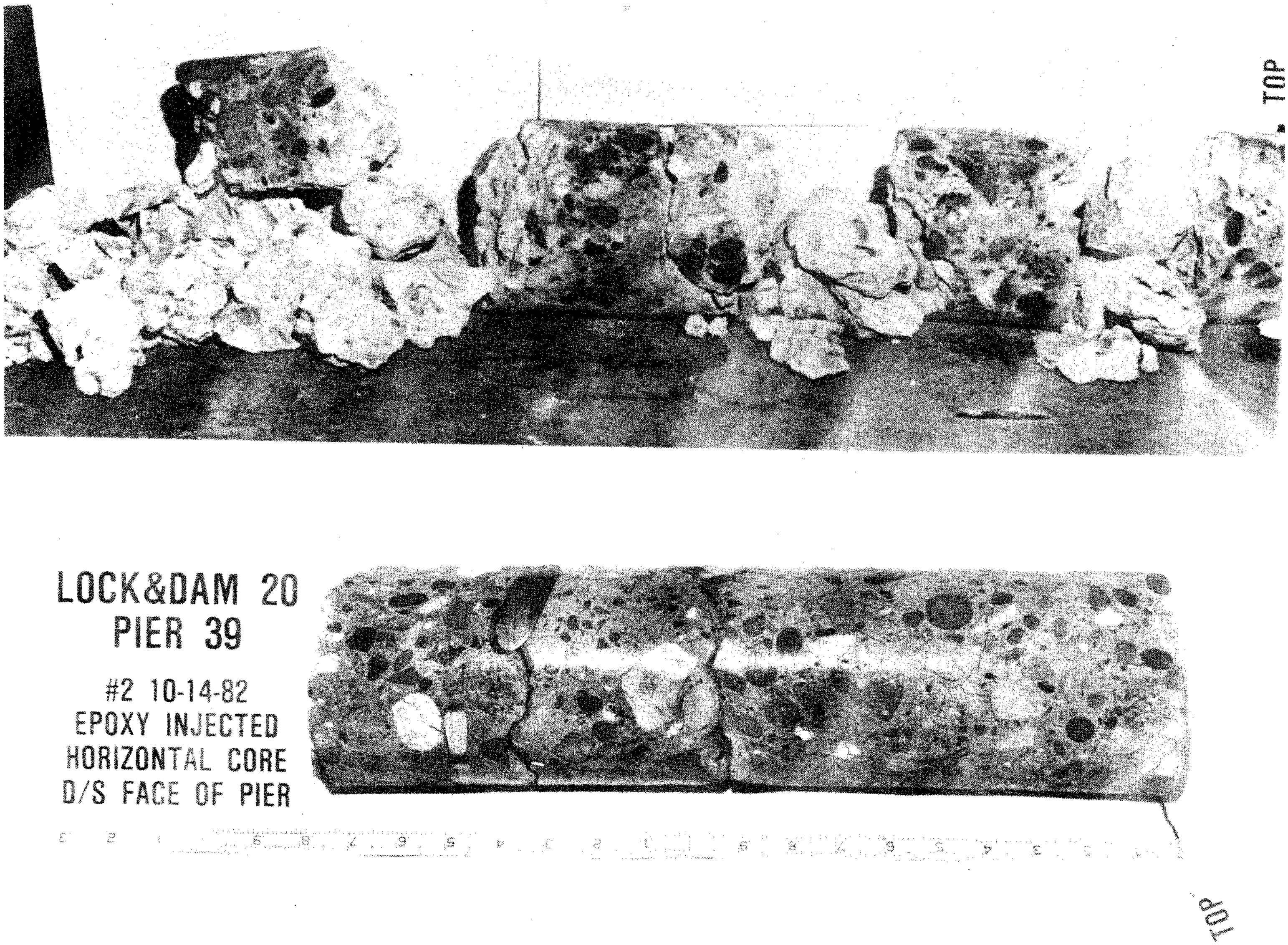


Figure 13. Cores taken from Pier 39 before and after epoxy injection.

The cores taken from the downstream face were not as severely cracked as the core removed from the pier top. The cracks were finer and more randomly distributed throughout the length of the cores. Fifty to ninety-five percent of the cracks were filled with epoxy. Tests for resistance to freezing and thawing subsequently run on two of these cores indicated the epoxy injection was successful in rebonding the concrete and in holding it together under freezing and thawing conditions. Most of the failure observed in these tests resulted from crumbling of the non-air entrained mortar and cracking of the coarse aggregate particles. In general, it was concluded that the epoxy in the fractures tended to hold the concrete together during the freezing and thawing cycles and aided in slowing down the deterioration of the concrete. Based on the tests, the performance of the concrete in the structure cannot be predicted absolutely; however, it is expected that while the exposed concrete surface would be subjected to damage due to freezing and thawing, the deeper interior concrete may be cemented together well enough to form an integral whole and to perform satisfactorily.

An inspection of the repaired pier stem, after one winter of service, however, indicated cracks in many areas of the epoxy surface coating on both the upstream and downstream faces of the pier as well as on the pier top. Leaching and calcium deposits were also present on the epoxy-treated surfaces as well as on some untreated surfaces. It was subsequently concluded that the deterioration observed in the pier stem was primarily due to continued lack of adequate drainage of rainwater from the top of the pier stem and a failure to adequately seal the pier top. Thus, water was still penetrating into and through the concrete, as evidenced by the calcium carbonate deposits that appeared after the completion of the epoxy injection work. In addition, it was concluded that the epoxies used to coat the pier were much too brittle, as evidenced by the reflective cracking.

From the experience gained at Lock and Dam 20, Flock and Walleser (1983) made the following recommendations regarding any future epoxy injection work.

(1) A visual survey of the damaged concrete should be made before locations for installing ports are selected. It is generally recommended that ports should be spaced at 1 to 1-1/2 times the depth of a crack. However, if a crack extends through to both sides, a grid of ports should be drilled to a depth of 1 ft 3 in. from the outside of the piers and staggered with ports drilled to the same depth from inside the pier arches.

(2) Water pressure tests should be made separately from the epoxy injection contract to estimate the severity of damage and to determine the location of ports.

(3) A phase-injection, multi-pump system should be considered. This system would consist of drilling grids of ports to different depths.

(4) Sandblasting is generally thought to be the most effective method of cleaning concrete surfaces. This belief proved to be true at Pier No. 39. The contractor attempted to remove incrustation by water jetting, but was unsuccessful.

(5) Wide cracks, such as those on the downstream face of Pier No. 39 where leakage occurred during injection, should be more positively sealed. Consideration should be given to "V" routing of these cracks and filling the "V" sections with an epoxy mortar.

(6) Injection packers must also be firmly sealed to withstand hydraulic pressure. The contractor used removable packers at Pier No. 39. A better method would be to use permanent-type packers which could be embedded deeper into port holes and cut off flush with the concrete after injection.

(7) In general, it would appear advisable not to use a filled epoxy for sealing the surfaces of the piers. If a rougher surface texture is required for paint, the epoxy seal can be scarified or even removed by heating to above 572°F.

(8) Provision should be made for repairing spalls and popouts. A filled epoxy should be used.

(9) The epoxy "pushed" or displaced water as it was injected into the pier. More positive relief could be provided by ensuring that the proper number and grid pattern of ports are selected prior to epoxy injection.

(10) Guideline specifications provided by epoxy resin formulators are basically applicable only to well-defined, two-dimensional cracks. The criterion for evaluating a satisfactory repair is that a crack be 90% filled. There is no direct correlation of this criterion to the three-dimensional, discontinuous type of cracking which is occurring in piers at Dam 20.

(11) Proper mixing, surface preparation, application, and selection of epoxy formulations are essential for good performance of concrete repairs. Furthermore, the repair at Dam No. 20 is not entirely applicable to present formulation guideline specifications. Therefore, reputable and experienced contractors familiar with current techniques should be employed.

5. Equipment, Labor, and Economics. The major piece of equipment required in pressure injection work is the pumping system used to inject the adhesive into the cracks. The injection process normally uses two positive displacement pumps geared together to provide the proper ratio for the adhesive components, an electric or air motor drive for the pumps, and a static mixing head. The exit nozzle of the mixing head is held against the face of the crack at entry ports which have been created by leaving an interruption in the surface-applied seal. For uniform surfaces and cracks which require relatively low injection pressures, 100 psi, a hot-melt thermoplastic seal can be used. In other cases, a cementitious, polyester, or epoxy seal is used. For cracks requiring very high pressures, 200 to 300 psi, to achieve penetration, a pipe fitting may be bonded into a hole which has been drilled with a hollow core drill to intersect the crack.

Other pieces of equipment may include a sandblasting unit for cleaning the exterior surface prior to sealing, a spray application unit for applying a seal coat to exterior surfaces, drills for inserting the injection ports, and assorted mixing and clean up supplies.

Labor required for any given job will be totally dependent upon the size and magnitude of the work to be done. Small jobs can probably be handled by a work crew of four or five men, whereas large jobs may require two or three times that number.

A unit cost for epoxy injection work is difficult to determine, since each job is bid individually on the basis of the type of structure to be repaired and the unique problems associated with that job. Material costs can be roughly determined by multiplying the number of linear feet of crack to be repaired by a factor of \$2 to \$4, depending upon the width of the crack to be repaired. This figure can then be doubled to obtain an estimate for the cost of labor.

Polymer Impregnation

Polymer impregnation has been used to rehabilitate a variety of portland-cement concrete structures including a structural floor slab in the Cass County Jail, Fargo, North Dakota; the deck of the Greenport Bridge, Greenport, New York; and an outlet tunnel wall and part of the stilling basin floor at the Dworshak Dam, Orofino, Idaho.

1. Cass County Jail, Fargo, North Dakota (1976) (Kaeding, 1976 and 1978). Built in 1913, the three-story reinforced Cass County Jail was condemned in 1974 because of its severe deterioration. Examination of the structure had revealed extensive cracking in the beams, columns, and walls as well as pockets of soft and weak concrete; layers of laitance within the walls; chemically induced deterioration; areas of segregation; weak, unwashed, and ungraded aggregates; and foreign material in the matrix. A load test, however, verified that except for the attic floor slab, the slabs on the occupied floors met the load capacity criteria of the American Concrete Institute's (ACI) building code requirements for reinforced concrete.

The 5-in. attic slab, which was an integral part of the structural frame, exhibited much more severe deterioration than the rest of the structure. Attempts to remove cores from the attic slab failed when the concrete disintegrated in the coring machine. This slab was repaired

using polymer impregnation after other methods of repair were judged unsuitable. Removal and replacement of the slab were not attempted because of the overall weakened condition of the remainder of the structure, which would have created the danger of total collapse of the structure during large-scale demolition.

The polymer impregnation process usually consists of four steps: (1) drying the concrete to remove free moisture present in the pore system, (2) cooling the concrete after drying, (3) impregnation of the concrete with a low-viscosity liquid monomer system, and (4) *in situ* polymerization of the monomer.

Because of the highly porous condition of the concrete in the slab, a void volume of about 27%, and low moisture content of 1.6%, it was unnecessary to dry the slab prior to impregnation. In fact, a preliminary, small-scale impregnation test on the slab indicated that the monomer penetrated the concrete so easily that it would be necessary to coat the bottom of the slab with a membrane to prevent the monomer from leaking out of the bottom of the slab.

The procedure used to impregnate the attic slab was as follows. The slab's surface was first swept clean to remove any dirt and debris. A 1/4-in. sand layer was then placed uniformly over the slab to hold the monomer in place during the soaking period. A pipe manifold system with spray nozzles was then constructed over the top of the sand layer for use in spreading the monomer over the sand blanket. A sheet of polyethylene was then placed over the area to be impregnated, to provide a vapor control cover. While this work was being performed, the bottom of the slab was sandblasted and then coated with an epoxy compound to provide an impervious membrane.

A monomer system consisting of 89.5 wt% methyl methacrylate (MMA) - 10 wt% trimethylolpropane trimethacrylate (TMPTMA) - 0.5 wt% azobis isobutyronitrile (AIBN) was used to impregnate the slab. The monomer system was applied to the sand blanket in four successive passes, with enough time between each pass for the monomer to soak into the concrete. A total of 3200 gal of monomer was used to impregnate the 7096-ft^2 attic slab at an impregnation rate of 0.125 gal/ft^2 per hour.

Once the slab had been impregnated, the sand blanket was removed and electric resistance heating elements were placed above the surface of the slab. The slab was then heated for 18 hours to polymerize the monomer. The temperature at the top surface of the slab was maintained at 170°F, while the temperature at the bottom surface of the slab was held at 145°F.

Upon completion of the polymerization cycle, the slab was cored and tested to determine the change in compressive strength. A final compressive strength of approximately 3000 psi was obtained for the impregnated concrete as opposed to an initial compressive strength of less than 800 psi.

In addition to rehabilitation of the attic floor slab, a number of other repairs were carried out to rehabilitate other structural members for a total cost of $887,343. The polymer impregnation work cost approximately $180,000. Had it been possible to remove and replace the attic slab, the estimated cost was about $42,000. The only alternative to restoring the structure was to demolish and replace it at an estimated cost of $4 million.

2. Greenport Bridge, Greenport, New York (1977-78). Polymer impregnation was used to restore the structural integrity of the badly deteriorated concrete deck of the Greenport Bridge, located on Route 25 between the villages of Greenport and Southold, Long Island, New York (Fontana and Kukacka, 1979).

The Greenport Bridge is a skewed, two-lane, three-span through girder with transverse floor beams and a reinforced concrete deck structure, built in 1929. The 3800-ft^2 bridge deck was originally a two-course construction deck with an 8-in. structural slab and a 4- to 6-in. wearing course separated by a bituminous membrane. At the time of rehabilitation, the deck was so badly deteriorated that it was impossible to remove an intact core.

The deck was impregnated one lane at a time, with each lane being divided into three sections. In general, the procedure used to impregnate each section was as follows. The wearing course and bituminous membrane were first removed and replaced with a temporary steel deck at roadway grade so that traffic could be maintained during impregnation of the structural slab. The steel plates used for the temporary deck were provided with a skid-resistant surface composed of silica sand bound with a polyester resin. Once the temporary steel deck was in place, the structural slab was dried by electric infrared heaters located beneath the deck. An enclosure consisting of semirigid fiberglass insulating board backed with aluminum foil was installed directly beneath the heaters to reduce heat loss to the structural slab.

Before the deck was dried, the underside of the bridge slab and the floor-beam and girder encasement concrete were sandblasted. Once the drying cycle was complete and the deck was allowed to cool, the underside of the bridge slab was coated with a polyester seal coat. The coating was done after the drying cycle was completed and before the impregnation cycle was started, to prevent thermal degradation of the sealant material.

The deck was then impregnated with a monomer system of 95 wt% methyl methacrylate (MMA) - 5 wt% trimethylolpropane trimethacrylate (TMPTMA), to which 0.5 wt% initiator azobis isobutyronitrile (AIBN) was added. The monomer was applied to the deck through a distribution system located between the temporary steel deck and the graded aggregate which had been placed over the structural slab once the wearing course was removed.

The rate of monomer application was dependent on the rate of absorption of monomer into the deck. The total impregnation time for each section was three to four days. Once the deck was saturated, the monomer was polymerized by reheating the deck using the infrared heaters.

After impregnation of the structural slab, the temporary steel deck was removed and wearing courses were placed to roadway grade, with polymer concrete used in the westbound lane and an asphaltic concrete in the eastbound lane. Several difficulties arose during the first drying, impregnation, and curing cycles. The temporary deck was not watertight, and water carried onto the deck by traveling vehicles seeped through the plates and resaturated the concrete. This resaturation substantially prolonged the drying cycle. During the first impregnation cycle, the monomer leaked through the polyester seal coat, which was ineffectual in retaining the monomer in the deck. The leaking monomer and some possible malfunctioning electrical component caused a monomer fire during the first curing cycle.

A reevaluation of the construction procedures delayed the project for several months until revised procedures could be instituted. The new procedures included use of a flexible silicone rubber coating to replace the brittle polyester coating and use of a gaseous carbon dioxide (CO_2) suppression system to eliminate the possibility of a monomer fire. The silicone rubber coating greatly improved the retention of the monomer in the structural slab, but because of irregularities in the bottom surface of the slab, it did not form a continuous film, and some monomer leakage was still evident. The CO_2 suppression system worked very well to eliminate fires on top of the deck. The remaining drying, impregnation, and curing cycles proceeded without major problems.

Once the deck was fully impregnated, full-depth cores were removed to evaluate the success of the repairs. As stipulated in the contract specifications, the impregnated structural slab was to be restored to the concrete slab's original design strength of 3000 psi in compression or higher. Results of the compressive strength tests indicated that the polymer-impregnated concrete (PIC) had an average compressive strength of 4320 psi.

The polymer loading in the deck, measured by thermogravimetric analysis, was found to vary between 3.1 and 17.5 wt%. This large variation can be attributed to differences in the quality of the concrete throughout the deck, not to differences in the impregnation procedure.

The water absorption of the PIC varied between 0.5 and 1.5 wt%. Two cores were subjected to 50 cycles of freezing and thawing, as per ASTM C-666, with no apparent loss of weight.

In general, all cores were well bonded and the polymer seemed to be completely dispersed throughout the core.

When evaluating the costs of the Greenport project, it must be remembered that the purpose of this experimental project was to test the feasibility of the proposed construction procedures for reconstituting deteriorated bridge decks in high density traffic areas by using polymer impregnation and to evaluate the long-term performance characteristics and the cost effectiveness of such repair. The actual cost of the impregnation work was $809,670. This was $40,404 over the contractor's original bid price. The increase in cost was due to a change in orders after the project had been started. The original engineer's cost estimate was $585,547. For this project, the rehabilitation of the structural slab by impregnation was 70% higher than the estimated cost of $475,000 to replace the structural slab by conventional means. However, for a comparable project in a high-traffic area such as New York City, the costs of the two methods would probably be much closer together. Building a full-service, temporary structure would necessitate condemning buildings, acquiring of property, and constructing a full-size structure. Since basic impregnation costs should not increase at a comparable rate for this much larger project, rehabilitation by impregnation would probably be cost effective in urban areas.

3. <u>Dworshak Dam, Orofino, Idaho</u> (1975). Polymer impregnation techniques were used to repair major cavitation/erosion damage in a portion of the stilling basin floor and the walls of an outlet conduit of the Dworshak Dam (Murray and Schultheis, 1977; Schrader, 1978; and Schrader and Kaden, 1976).

The Dworshak Dam is a straight gravity structure, 717 ft high, with a crest width of 3287 ft. The dam contains three similar regulating outlets that are 12 ft wide by 17 ft high.

The outlets, first used in 1971, had been used intermittently for a total of 10 months before being repaired. The outlets were inspected in June 1973, at which time some minor, isolated surface scaling was noticed. A year later, two outlets showed severe cavitation damage 50 to 75 ft downstream of the outlet gate. Severe damage was described as massive removal of concrete and removal of reinforcing steel. The worst area had a depth of concrete failure of approximately 22 in., and some of the No. 9 reinforcing bars were missing. Downstream of the severely damaged areas in Outlet No. 1, medium damage, less than 1 in. in depth, totaling over 60 yd^2 of surface area was found throughout the remaining 200 ft of the outlet. Surrounding these areas of medium damage were areas of light erosion or surface scaling as well as areas where the original curing compound was still intact. Every horizontal construction joint in the outlet showed typical scaling and the beginning of failure.

The stilling basin received severe damage which removed approximately 1600 yd^3 of high-strength reinforced concrete from its 29,261 ft^2 floor surface. The damage was primarily attributed to debris and gravel trapped in the stilling basin. Damage varied from 0.1 to 9 ft in depth.

The procedure used to repair the walls in Outlet No. 1 were as follows. Severely damaged areas were repaired by outlining them with a 3-in.-deep saw cut and then removing all the damaged concrete within the saw cut to a minimum depth of 15 in. Reinforcing bars were reset within the chipped out areas, which were then filled using fibrous concrete.

Areas of medium damage were bush hammered back to a depth of 3/8 to 1 in. to sound concrete and then the voids were filled with "dry packed" concrete mortar. Once these repairs were completed, all wall surfaces in Outlet No. 1 were impregnated to a height of 10 ft.

The concrete in the outlet tunnel walls was impregnated in 10- by 12-ft sections using the following procedure.

The concrete was dried using infrared heaters mounted inside 12- by 14-ft insulated plywood boxes. The boxes were placed against opposite walls and braced against themselves during the drying operation. The concrete was dried at a surface temperature of 275°F for a period of 5 hours, after which the concrete was allowed to cool to a surface temperature of 90°F. Once the wall had cooled down, it was impregnated with a monomer system consisting of 95 wt% MMA - 5 wt% TMPTMA - 0.5 wt% AIBN.

Soaking was accomplished using two 10- by 10-ft stainless steel panels, which were pressed against the outlet walls to form shallow tanks to contain the monomer. The panels were mounted on a cart, which was pulled up the outlet by cables. The gasket around the perimeter of the panel provided about a 1/8-in. gap between the wall and the panel. To assure that this gap was maintained throughout the soak area, spacers were tack welded to the panel surface. A bead of caulking was then placed around the outside panel edge next to the gasket, and waterproof tape was placed over that with half the tape on the concrete and half on the panel edge. This seal, however, did not prevent leakage problems at construction joints and open structural cracks. It was eventually determined that leakage could be controlled by applying a coat of gel epoxy to the concrete wall surface where the perimeter gasket made contact. When open construction joints and wide cracks were encountered, they were injection grouted with epoxy at the perimeter of the soaking panel.

The soaking panels were filled through a manifold at the bottom of the stainless steel plate. The manifold was connected to monomer storage tanks which were pressurized to force the monomer into the soaking chamber. A sightglass located at the top of each chamber was used to determine when the chamber was filled. By maintaining pressure on the storage tanks, monomer was continually added from the reservoir at the same rate that it soaked into the concrete. The concrete was soaked for 6 hours. This was sufficient time to impregnate the concrete to a minimum depth of 0.5 in.

After completion of the soaking period, the excess monomer was drained from the chambers back into the storage tanks. The chambers were then filled with hot water to polymerize the monomer. Electrical resistance heaters were installed on the back of the soaking chambers to elevate and maintain the water temperature. The backs of the panels were insulated to reduce heat loss and to protect the workmen. As a safety precaution, the heaters were not activated until the chambers were filled with water and no explosive vapors were present. The water temperature was maintained between 150°F and 210°F for a minimum of 2 hours. The polymerization procedure was then complete; the water was drained from the forms, and the forms were moved. This procedure was repeated until the outlet walls were impregnated.

During the time the impregnation work was being performed in the outlet tunnel, a vapor monitor with a remote sensor continually indicated the atmospheric concentration of MMA vapor. A continuous strip chart recording of these readings was maintained to give an accurate history of vapor concentrations in the outlet at all times. When the vapor concentration reached 100 parts per million, a visual and an audio alarm were activated, at which time personnel were required to put on protective clothing and face masks, work was to stop, and steps were to be taken to decrease the concentration. In actuality, the vapors generally stayed at very low levels, in the range of 3 to 15 parts per million. The only exception to this was during the first soak when a bad leak occurred at a construction joint before the procedure for grouting those joints had been established.

The erosion damage in the stilling basin floor was filled with a conventional concrete mixture to within 15 in. of the final floor surface. This was topped out with a 15-in. overlay of fibrous reinforced concrete mixture anchored with No. 8 bars. One half the floor was then surface impregnated.

Concrete in the stilling basin was impregnated in 58- by 12-ft sections. An enclosure with insulated plywood sides and a removable top was placed over each area to be treated. Before the surface was dried, free water was swept away or vacuumed up and a bead of caulking was placed around the base of the enclosure to keep water from reentering during the drying cycle. A 3/8-in. layer of sand was then spread over the surface to act as a wick during soaking and to promote better distribution of the monomer. Infrared heat lamps spaced at 24-in. centers and 18 in. above the floor were installed in the enclosure, and a lightweight insulated roof was placed over it. The lamps used a total of 93 kilowatts or 134 watts per square foot and normally accomplished the drying in 10 to 15 hours. At a thermostat setting of 450°F for the air inside the enclosure, a surface temperature of 220°F could be attained in about 2 hours. With the thermostat at that setting, the surface stayed at 220°F to 320°F for the remainder of the drying cycle. Moisture was allowed to escape through small openings spaced about every 5 ft in the enclosure walls. Temperatures at different depths within the concrete were measured during the drying process of the first setup using embedded thermocouples that had been cast into the fibrous concrete slab. After drying, the concrete went through a controlled cooling process. The heat lamps were turned off, but the roof sections for the enclosure were kept in place. After a brief period, the roof panels were cracked open about 1 in. After a total cooling period of about 6 hours, the surface temperature was about 120°F. The roof sections were then removed, and the monomer was applied.

The monomer system used to impregnate the floor of the stilling basin consisted of 97.5 wt% MMA - 2.5 wt% TMPTMA, with 0.5 wt% AIBN initiator.

The monomer was applied with a 12-ft long sprinkler pipe, which was moved back and forth above the surface to be impregnated until a predetermined amount of monomer had been applied. The monomer flowed by gravity from an elevated mixing drum, through a hose, and into the sprinkler pipe. A polyethylene sheet was draped over the surface to reduce the evaporation losses, the roof sections were reinstalled, and another polyethylene sheet was laid over the entire enclosure to keep down any escaping fumes. In the first application, a total of 55 gal of MMA was

applied. The dosage was gradually increased to about 80 gal per setup (0.115 gal/ft^2), with which the sand blanket would just begin to stick to the floor, indicating that the concrete could not absorb any more monomer. Since application of the monomer began when the surface temperature was near the point at which polymerization could start, the percentage of cross-linking agent was decreased. This adjustment made the monomer slightly more difficult to polymerize because the rate of reaction was slower in the temperature range of about 110°F to 140°F. However, since very hot steam was used to polymerize the stilling basin floor, obtaining complete polymerization was no problem. After a usual soak of 6 hours, wet steam at 750°F was used to polymerize the concrete. The portable steam generator used for this was fed by an air hose at 115 psi pressure. The steam was distributed through a manifold in the center of the enclosure, and within a minute, the entire area being polymerized was uniformly filled with steam. After 1-1/2 hours, the steam was turned off and the process was completed. Total time for both the impregnation and the polymerization cycle in the stilling basin was normally 24 hours, excluding initial setup and final cleanup.

Upon completion of the impregnation work, cores were taken from both the outlet tunnel walls and the stilling basin floor to evaluate the effectiveness of the impregnation process and the depth of impregnation. The cores typically indicated that impregnation depths varied between 3/4 and 1-1/4 in. for the outlet walls and 1/2 in. for the stilling basin floor. Cores taken through the dry-pack patches in the outlet wall indicated that the polymer had penetrated completely through the dry pack material and into the base concrete.

During impregnation of the stilling basin floor, moisture kept migrating up through vertical construction joints and cracks. It was, therefore, impossible to dry the concrete at these locations. In most cases, the moisture evaporated as it came through the joint or crack, and floor surfaces that were more than about 6 in. from them were adequately dried. Consequently, cores taken at cracks and joints showed no polymer impregnation, while cores away from the damp areas had been successfully impregnated.

During the impregnation work, a number of random cracks were noticed in the fibrous concrete floor. When the surrounding areas were dry, the cracks showed a dark trail of moisture and became very apparent. These cracks were not caused by the drying process. They were present in the floor before drying but were not as noticeable until the surface was cleaned and dried. After the polymerization cycle was completed, there were no new cracks and there was no apparent growth of the old cracks. If moisture could have been kept out of the cracks, they would have been filled and structurally sealed with polymer.

The outlet walls absorbed monomer at a rate of about 0.3 gal/ft^2, regardless of whether the area was dry-pack patching, new fibrous concrete, or the original conventional concrete. The impregnation depths were also similar in each of the three materials. The stilling basin floor absorbed all the monomer when it was applied at a rate of 0.086 gal/ft^2. When monomer was applied at a rate of 0.115 gal/ft^2, a small amount of monomer remained at the surface. It is difficult to explain why the outlet walls soaked in more monomer and correspondingly had greater depths of impregnation than the stilling basin floor, but the reason can undoubtedly be related to at least one of the following: (1) Moisture was being forced up through the floor of the concrete stilling basin by uplift pressure. This made drying more difficult and refilled the microvoids with water between the depths of 1/2 to 1 in. during the cool-down cycle. During the soak phase, monomer could not enter the concrete below 1/2 in., since it was already saturated with water. (2) The cool-down period was not sufficient to allow concrete 1/2 in. deep or greater to reach the temperature where polymerization would not be initiated. During the soak cycle, monomer would penetrate to a depth of about 1/2 in., where temperatures were high enough to cause polymerization. The polymerized monomer then acted as a barrier, which would not allow unpolymerized monomer to soak into the concrete beyond that depth. Since the amount of cross-linking agent was purposely reduced to help avoid this situation, its occurrence is unlikely. (3) The drying duration and/or temperature was insufficient to dry

concrete deeper than 1/2 in. For most of the drying time, the temperature was probably 260°F to 300°F. Durations of drying for the stilling basin were generally on the same order as for the outlets except that some setups were a little longer and some were a little shorter.

The dam was inspected several months after the work had been completed and after the dam was back in service. The inspections indicated that the PIC has held up very well, while some of the other repairs made to the structure, such as epoxy mortar repairs, have not.

4. Equipment and Economics. As can be determined from the case histories cited above, a large inventory of specialized equipment is necessary to impregnate a concrete surface successfully. Although specific equipment needs depend upon the nature and requirements of each individual project, general needs include the following: an insulated drying enclosure, heaters for drying the concrete, temperature recorders for monitoring drying and curing temperatures, an impregnation enclosure (required for vertical applications), monomer storage and distribution equipment, monomer mixing equipment, caulking and sealing compounds, a curing enclosure, heaters for curing the impregnated concrete, vapor monitoring equipment, fire extinguishers, and safety equipment for individual workers, such as rubber gloves, boots, impervious aprons, and respirators.

Monomer requirements depend upon the quality of the concrete to be impregnated and the desired depth of impregnation. For good quality concrete, impregnated horizontally, a monomer application rate of 1 gal/ft^2 results in a depth of impregnation of approximately 0.5 in.

It is equally difficult to determine a unit cost for polymer impregnation work since each project is usually very different from the previous one. A wide range of costs have been reported for impregnation, varying from \$7/ft^2 for the work done at the Cass County Jail, to \$213/ft^2 for the work done at the Greenport Bridge. This variation is partly caused by the fact that contractors are unfamiliar with what may be expected of them, and the wish to pay the cost of new equipment on a single small job. However, costs should stabilize as contractors become more familiar with the process and acquire the necessary equipment.

When polymer impregnation is being considered as an alternative to other forms of repair, the cost and time lost in shutting down the facility must also be considered. In many cases, impregnation work can be scheduled and performed while a facility is being operated, with only the immediate work area closed to operations.

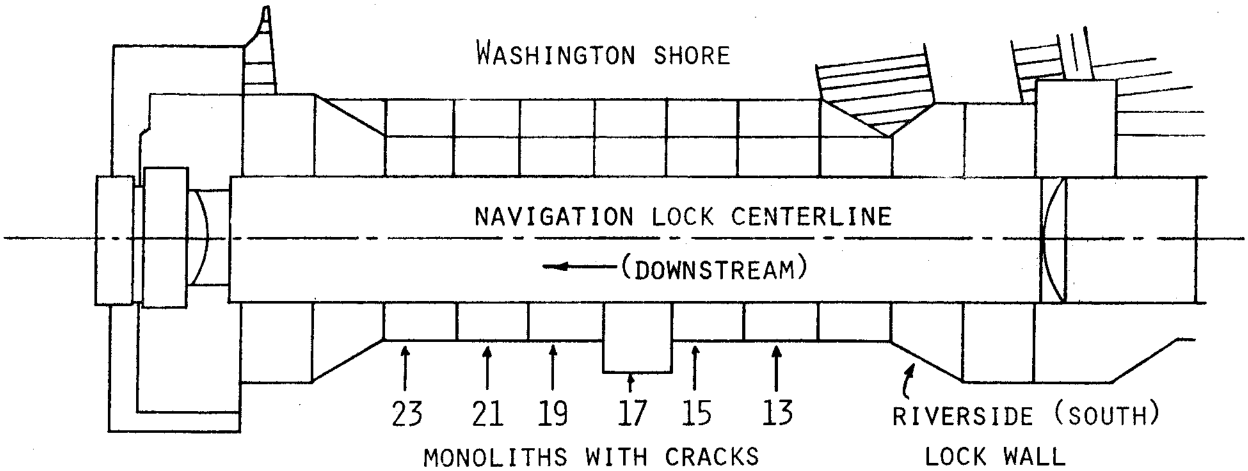
Addition of Reinforcement

The addition of reinforcement to concrete structures has been used in a variety of applications to help seal cracks and to strengthen under-reinforced structures. Presented below are case histories of four such applications.

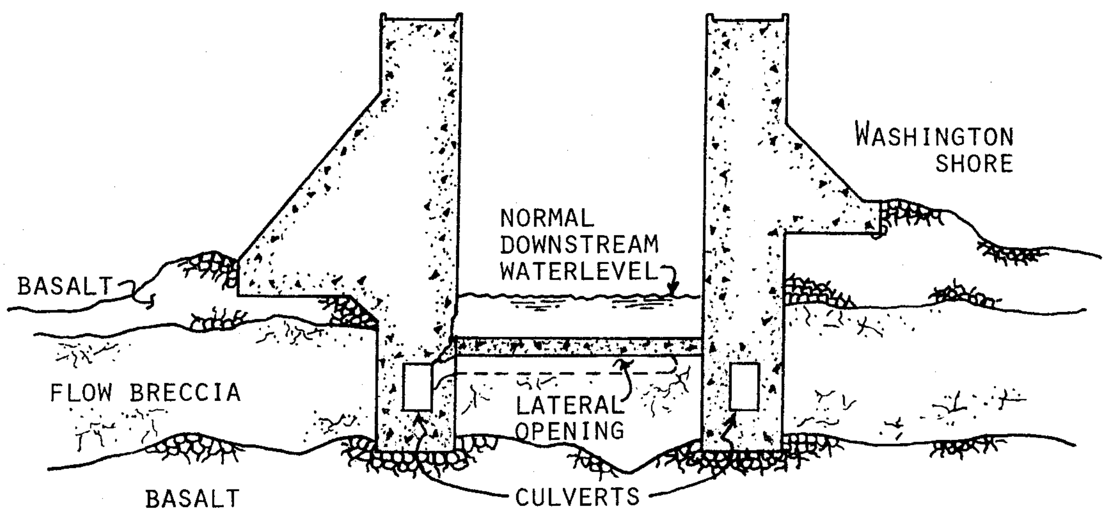
1. John Day Navigation Lock and Dam, Oregon (1981). The John Day Lock and Dam is located on the Columbia River between Oregon and Washington about 110 miles upstream from Portland, Oregon. The lock is 675 ft long, 86 ft wide, and provides a maximum lift of 113 ft (Figure 14). The lock began operation in 1968.

During an inspection in 1975, significant structural cracking and related spalling were discovered in two of the lock monoliths, monoliths 17 and 19. By 1979, cracking and spalling had been detected in four additional monoliths, monoliths 13, 15, 21, and 23. The structural distress in the monoliths consisted of cracks that originated at and propagated from the upper inside corner of the filling and emptying culvert and terminated at the surface of the wall in the lock chamber. Because of the continued progression of the cracking and spalling, a series of remedial repair procedures were devised and implemented to halt and correct the ongoing deterioration (Adhesive Engineering Bulletin, 1981b, Barlow, 1986, and Neuberger, 1982).

The objectives of the selected repair procedures were to restore the existing cracked structure to near its original uncracked condition, to eliminate the cause of the cracking, and to effect the repairs without



A. Plan view of navigation lock



B. Cross-section of structure (looking downstream).

Figure 14. John Day Navigation Lock.

significantly affecting the operation of the facility. This was accomplished using a three-phase repair program. The program consisted of: (1) grouting the foundation rock, (2) installing rock anchors through the cracks after injecting structural epoxy adhesive into the cracks and, (3) repairing the surface of the wall in the lock chamber (Figure 15).

A stress analysis using finite element models revealed two probable causes for the cracking. One cause was due to the original procedure used to fill and empty the culvert that, before being changed, produced high hydraulic surge pressures in the culvert. The other cause was due to the normal lock-full condition which produced excessive foundation deformations because of a layer of weak-flow breccia rock. Both causes produced high tensile stresses in the concrete and, because of inadequate reinforcing steel in the cracked areas to distribute these stresses, led to the ensuing cracking and spalling. In addition, the cyclic loading of the lock walls, due to filling and emptying of the lock, contributed to the continued propagation of the cracking and spalling.

In Phase One of the repair program cement grout was pumped into the flow breccia sandwiched between the two layers of basalt to fill in any voids or open joints within the rock mass. The cement grouting was done in several stages beginning in 1980. Post-consolidation grouting deflection measurements at the crest of the monoliths revealed that deflections were reduced 50 to 60% from the pregrout condition.

In 1981, Phase Two of the repair program was performed. This phase of the repair program involved the installation of 73 rock anchors and the injection of structural epoxy adhesive into the crack network. Finite element analysis had shown that the forces induced by the rock bolt anchors would substantially reduce the tensile stresses in the cracked areas at the top of the culvert.

Subsequent to the grouting of the foundation, 10-in. diameter rock anchor holes were drilled from the outside of the monoliths into the basalt rock below. This work was carried on without interrupting lock

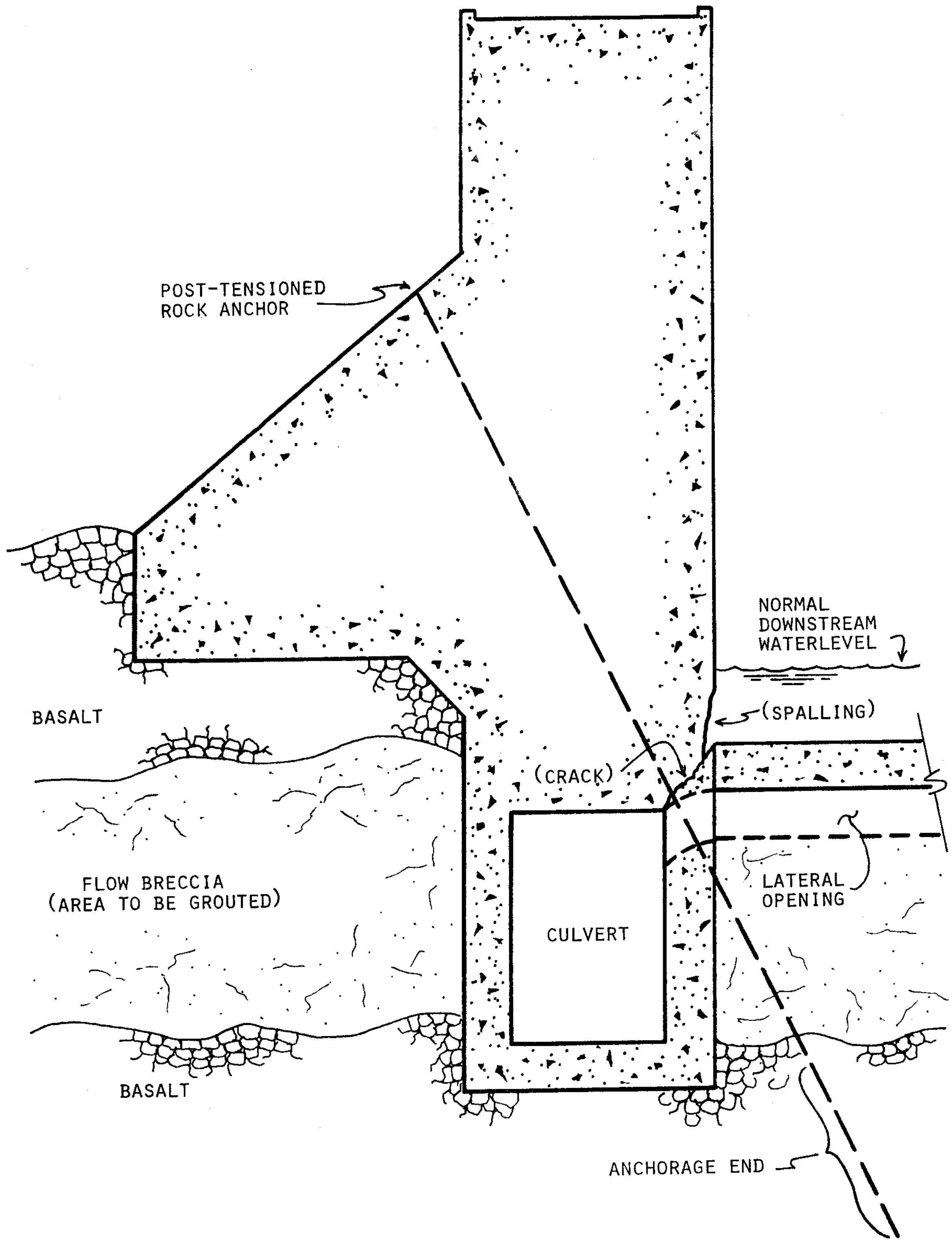


Figure 15. Cross-section of lock wall, showing details of crack location and supporting rock mass.

service. The holes were drilled at either 55° or 66° entry angles from the horizontal. The allowable deviation to avoid drilling into either the filling and emptying culvert or the lock chamber was 1 ft in 100 ft. The holes varied between 138 and 172 ft and terminated approximately 40 ft below the base of the lock.

The 73 rock anchors consisted of 37 seperate, 7-wire strands, 0.6 in. in diameter. The rock anchors consisted of four major elements: the 30-ft anchor ends, the inflatable grouting packers above the anchor ends, the greased and polyethylene sheathed strands above the packers, and the stressing end assemblies. Packers were used to assure a good grouting job in the crucial anchor end area. Greased and sheathed tendons were used to have the capability to retension or detension the rock anchors. The rock anchors were generally spaced at 4 ft intervals without any disruption to lock service. The 30-ft anchor ends were grouted during slack times in lock usage.

Following the placement and grouting of the rock anchors, the lock was shutdown for 30 days and dewatered so that the cracks in the monoliths could be epoxy injected and the subsequent post-tensioning and grouting of the anchors could take place.

In order to confine the cleaning fluids and ultimately the epoxy adhesives, each of the monolith joints had to be isolated to stop migration upwards and downwards of the cleaning solution. This was done by drilling 6-in. diameter core holes from the lock wall through the monolith's joints intersecting the culvert. The holes were positioned above and below the primary and secondary cracks that intersected the joints. The core holes were then backfilled with a rapid setting cementitious, flowable, non-shrink grout to provide a plug.

The monolith joint faces were then sealed and ported for injection. A 100% solids, epoxy paste adhesive was then injected into the joints, with a single component piston pump, from the culvert until it vented on the lock wall side. The consistency of the epoxy was such that it penetrated the joint with a minimal amount of material migrating into the cracks intersecting the joints.

To provide maximum penetration of the cleaning solution, approximately 3300 lin. ft of 1-1/2 in. diameter entry ports were drilled into the lock wall, approximately 4 to 6 ft on center, to intersect the interior cracks. Once the holes were drilled, they were flushed to remove any remaining drilling slurry. Deformed bars with epoxy grout tubes were then inserted into the holes. The bars acted both as a filler to reduce the volume of epoxy injected into each hole and as reinforcing to transfer stresses across the cracked section. The tubes acted as filling and venting ports for the cleaning solutions and the epoxy adhesive.

The entire crack length of 440 ft, extending throughout the six monoliths in the culvert were then sealed and ported using an epoxy adhesive. A bio-degradable alkaline-based detergent was then introduced into the crack network through a manifold system to flush out river silts and clay which can have a detrimental effect upon obtaining a good structural bond. Incorporated into the cleaning solution was a dye to help locate any leaks or breaks in the crack and joint seals. The cracks were then flushed with clean water to remove the detergent and blown with air to remove excess water and any remaining particles.

Epoxy injection of the cracks began in the culvert. A high strength, creep resistant, rapid curing, low viscosity injection adhesive capable of bonding in wet conditions was used (Adhesive Engineering's Concresive 1380). Injection continued until material began to vent on the lockside wall vents. Injection then continued in the lock by injecting into the vent that initially showed material and progressed up the 1-1/2 in. diameter hole grid until the entire crack network was filled. Approximately 600 gal of epoxy was injected into the crack network, completely filling the primary and secondary cracks.

Two days after the injection was completed post-tensioning of the rock anchors began. Each tendon was stressed and 24 hr later checked again to insure proper tensioning. Coring from inside the lock chamber into the crack network was performed by the Corps of Engineers to verify filling of the crack network. The second phase of the repair program was completed in September 1981.

In March 1982, the dewatered navigation lock was inspected to evaluate the effectiveness of the Phase Two repairs. This was the first opportunity to inspect the repaired area since completion of the structural repair work. The filling and emptying culvert was inspected initially. A hairline crack was discovered in the epoxy patch at the culvert north wall-ceiling interface. The hairline crack was most evident in monoliths 21 and 23 where it was continuous throughout the length of the monoliths. Traces of the hairline crack were also seen in the other repaired monoliths (11, 13, 15, 17, and 19), but they did not appear to be continuous and were evidently only in short lengths. Because of no spalling or chipping at its edges, the hairline crack showed no signs of working or being progressive in nature. After the culvert was inspected, the lock chamber wall was inspected. Evidence of recracking of the injected epoxy was found only in a monolith 15 ladder well. The crack was hairline and did not appear to be progressive. No evidence of recracking was found at any other areas and no traces of the crack were seen at the epoxy patch areas where the previous cracking had surfaced. Areas of previously drummy surface concrete were found to be solid, indicating that the injected epoxy was bonding the cracked surfaces together.

In general, the repairs were considered to be successful. Even though there was evidence in the culvert of some hairline cracking of the injected epoxy, the hairline cracking had not progagated through the culvert wall and out to the lock chamber wall surface. In addition, the nonworking of the newly discovered hairline crack edges indicated that the 73 installed rock anchors were keeping the crack restricted and preventing any new large scale cracking and spalling. After 8 months of lock use following the completion of the structural repair, the repaired monoliths were in a stabilized condition and showed no signs of future continued cracking and spalling. The total cost of the repair program was approximately $3.5 million, a savings of approximately $900,000 over other methods that had been considered.

2. Markland Locks and Dam, Ohio River (1981). Markland Locks and Dam is located on the Ohio River approximately mid-way between Louisville, Kentucky, and Cincinnati, Ohio. The locks and dam were constructed in the late 1950's and early 1960's as a replacement of Lock and Dams 35, 36, 37, 38, and 39 which were constructed in the 1920's. Markland Locks and Dam project consists of two 110-ft wide locks (600 and 1200 ft long), a dam of twelve 100-ft wide tainter gates supported by concrete piers, and a hydroelectric plant. A two-lane state highway bridge also crosses the structure.

The locks which are situated on the left bank, are constructed of concrete gravity walls founded on rock, and incorporate steel miter gates. Filling and emptying of the lock chambers is accomplished through longitudinal culverts located within the lock walls. Flow of water through the culverts is controlled by tainter valves located in special "valve monoliths" located near the upstream and downstream end of each lock chamber. The tainter gate is located in the culvert at the bottom of the recess. Raising and lowering of the tainter gate controls the flow of water through the culvert and in the process, water rises in the shaft and the recess. Depending on the function being performed and the valve monolith involved, a differential head of up to 46 ft may exist between the water level in the recess and the lower pool. When the lock chamber is dewatered for maintenance work, the differential head may be as much as 55 ft.

In the early 1960's, a number of longitudinal cracks began developing in the valve monoliths. The cracks extended from the corners of the main recess to smaller recesses or to the joints at the end of the monolith. In addition, some cracks extended from the top of the lock wall to the top of the culvert. With hydrostatic pressure being exerted within the recess and only minimal transverse reinforcement in the ends of the monolith, the danger existed that the monolith could fail by splitting longitudinally. The risks and costs associated with such a failure were considered unacceptable, so repair of the monoliths was required (Keith, 1972).

Acceptable repair methods had to meet the following criteria. They had to be structurally adequate, constructable, cause a minimum of damage to the existing structure, provide for minimum disruption to river traffic during construction, leave no obstructuions to traffic after the job was completed and be implementable at an acceptable cost.

The monoliths were repaired by drilling holes horizontally into each end of the recess perpendicular to the face of the lock wall and installing high strength steel rods into the holes to resist the internal hydrostatic pressures. Dywidag threaded bar rock bolts were used and were anchored into place using Celtite polyester resin. The work was performed from a floating plant within the locks. This permited work at various levels by simply raising or lowering the water level within the locks. The work crew and barge were moved out of the lock at the end of a shift to permit passage of traffic in the main lock.

A track mounted, diesel precussion drill, located on the barge, was used to drill the holes into the valve monoliths. After the holes were drilled and cleaned, cartridges containing the Celtite polyester resin were inserted into the holes. Two types of cartridges were used. Cartridges containing a quick-setting polyester were inserted within the anchorage zone of the hole and cartridges containing a slow-setting resin were inserted into the stressing zone. After the cartridges were in place, the rock bolts were threaded into place using the drill. The threading action activated the curing of the polyester resin by mixing the initiator into the resin. Once in place and firmly anchored by the polyester resin, the bolts were stressed to 0.7 ultimate strength using a hydraulic jack and jacking chair. The bolts were held in place for a minimum of 4 hr with anchoring nuts to assure that the slow setting resin had fully cured. The nuts were then removed by flame cutting, and the holes were filled with a non-shrink grout to form a surface flush with the wall.

Work was started in August and completed in November 1981. A total of 240 anchors were installed at a cost of $170,000. Although some problems and difficulties were experienced by the contractor, the repairs were considered to be quite acceptable.

3. Apartment Building, Brussels, Belgium (1982). A gas explosion caused by human error took place in one apartment on the tenth floor of a 26-story apartment building in Brussels, Belgium. The apartment was almost completely enclosed by reinforced concrete walls which had been designed to take the wind loading of the structure; therefore, almost all the damage to the structure was limited to the apartment. Within the apartment, however, the damage was considerable.

The inner walls and glass panels were blown away by the force of the explosion. During the first phase of the explosion, the ceiling slab was loaded upward by the overpressure; during the next phase, it was loaded downward by the underpressure. As a result, the slab exhibited large deflections, with a maximum deflection of 2 in. In addition, the concrete at the surface of the ceiling slab was severely deteriorated, by a fire which arose following the explosion.

After a careful examination, it was decided that the damage could be repaired by removing the loose and unsound concrete and exposing a clean sound concrete surface. These areas would be repaired using an epoxy mortar, EPISOL-EMT (NV. Resiplast-Belgium). Steel plates would then be bonded to the underside of the slab to reinforce it and to eliminate the deflections caused by the explosion (Van Gemert and Maesschalck, 1983).

The deflections were eliminated using eight hydraulic jacks, mounted on a special supporting structure. Next to each jack was placed an adjustable screw. During and after the lifting operation these screws served to secure the slab in the lifted position. With the hydraulic jacks, the slab was put back to a nearly horizontal position.

At lifting, new cracks appeared at the top side of the slab. To prevent them from closing again when the supporting structure was removed, they were filled with a low-viscosity epoxy resin. At the edges, cracks

originated at the bottom face of the plate. These cracks were filled by pressure injection of epoxy resin. The steel plates were then glued to the bottom face of the slab with epoxy, EPICOL-U.

The steel plates had a cross section of 0.2 in. by 9.84 in. and were spaced at 31.5 in. The cross-sectional area of these plates was determined by the limitation of the deflections. Plastic deformations during the explosion disturbed the continuity of the slab over its supports so that deformability of the slab had increased. External reinforcement was applied, which increased the stiffness of the slab, so that the deflections remained within the limits allowed.

Special attention was paid to the design of the anchorage lengths of the plates. The maximum shear stress in the epoxy joint was, therefore, calculated. The maximum shear stress allowable in the joint corresponded to the surface tensile stress of the concrete. The surface tensile stress was measured by tear-off tests on small steel cylinders, glued to the concrete surface.

After a curing time of seven days, the epoxy glue attained its final strength, and the supporting structure was removed, at which time the slab underwent a deflection of 0.2 to 0.3 in. These deflections corresponded to the calculated values. The repairs were, therefore, considered successful.

4. Kansas Department of Transportation. For many years, the Kansas Department of Transportation (DOT) was faced with the problem of how to repair shear cracks in the girders of many of its two-girder reinforced concrete bridges. The Department had tried repairing some of the girders using epoxy injection, but this method was not successful since the repair procedure did not improve the shear capacity of the girder. Some girders were repaired by removing the sections of cracked concrete in the girder, adding reinforcing steel, and recasting the girder to its original dimensions. This method, however, requires the use of supporting falsework and necessitates the closing of part of the bridge while the work is being done.

As a result, an alternative method of repair was needed which would permanently repair the cracks and improve the girders' shear capacity at low cost and without serious traffic delays. Post-reinforcement, a repair method which meets each of these objectives, was subsequently developed and is now routinely used by the Kansas DOT (Stratton and Crumpton, 1984; Stratton et al, 1978).

Post-reinforcement, as developed by the Kansas DOT, consists of the following steps: (a) sealing of the surface of the crack using a silicone sealant, (b) vacuum drilling dust-free holes 6 in. apart and 45^{o} to the deck surface, thereby crossing the crack plane at 90^{o}, (c) filling the hole and crack plane with epoxy pumped under low pressure, and (d) placing an almost full-depth length of reinforcing bar (No. 4 or 5) into the drilled hole to span the crack by at least 18 in. The epoxy bonds the bar to the walls of the hole and fills the sealed crack plane, which bonds the cracked concrete surfaces together monolithically and reinforces the section. A detailed description of the repair process, as described by Stratton and Crumpton (1984), is presented below.

> Contractors are supplied construction plans and specifications for performing a post reinforcement repair. If the bridge is covered with an asphalt wearing surface, before drilling can begin, the asphalt must be removed. Cracks on the surface of the girders are sealed with an elastic silicone sealant. The clear sealant bead is applied over the crack and then pressure screeded into the crack with a specially shaped spreader. On wide cracks some buildup of sealant may be needed. To avoid trapping rainwater in the cracks, they are not sealed much in advance of the repair.
>
> While sealing is in progress, the drill entry points are marked on the bridge deck near the measured centerline of the girders. After the ideal design positions are marked, the actual entry points are established using a pachometer (metal detector) to find the transverse deck rebar. The actual drill entry position will usually be within 2 in. of the design position shown on the plans. Without the pachometer to locate the rebar, the drill crew stands a high chance of hitting the rebar, which would cause tip breakage and lost time.

Drilling should start at the center of the span and progress toward the pier. This plan avoids having the drilling rig set up over holes that are already drilled. To commence drilling, the trailer or truck on which the drill is mounted is centered over the entry point, and the drilling equipment is leveled side to side using hydraulic stabilizers. Next, the gantry that supports the drill is raised to a 45° angle, locked at this angle, and then moved so the drill tip is at the entry point marked on the bridge deck. A drill entry gauge can be spot-faced in the deck using a light chipping hammer. This method facilitates drill entry into the concrete and also reduces tip breakage. Once the tip is in position, the desired hole depth is marked on the drill steel. Drilling the hole generally takes less than 1 minute, and turn-around time from hole to hole is usually less than 3 minutes. The number of holes drilled in a day is limited by the number that can be injected with epoxy on the same day.

After being drilled, each hole is measured for depth, and a rebar is cut 3 in. short of full hole depth. This length keeps the top of the bar below the path of the grinder used to prepare decks topped with a thin bonded concrete overlay.

The epoxy injection crew starts work after drilling is completed on one girder part. Injection starts with the deepest hole, i.e., the hole closest to the pier or the abutment. After the hole is about half filled with epoxy, the bar is slowly inserted and gently tapped to be sure it is seated on the bottom of the hole. The nozzle is then reinserted, locked in place, and the hole is filled under pressure.

Any crack in the girder will likely be intercepted by at least one drilled hole. To fill tight cracks with epoxy, a sustained pump pressure of 100 psi is usually required. Less pressure is required to fill wide cracks. Consequently, the operator should not try to increase pump pressure to 100 psi when wide cracks are encountered. To do so might cause a rupture in the silicone material sealing the crack exterior. Instead, the operator should watch the air cylinder

shaft of the pump for any displacement. As long as the air cylinder shaft is moving and no leaks from the surface cracks are present, the crack filling operation is progressing satisfactorily. Pumping should continue until either epoxy is detected in the next hole or the air cylinder shaft stops moving.

The injection crew should constantly check previously injected holes, and if a lower level of epoxy is noted in the holes, they should be refilled. In very hot weather, coarse aggregate can be poured into the hole on top of the bar to cool the epoxy. Otherwise, the heat of polymerization may cause the epoxy to boil and foam, or it may cause thermal contraction cracks at the epoxy surface.

To perform the post reinforcement repair, a contractor must have certain equipment. The 1-in. diam holes must be clean, dust-free, and dry. The depth of holes must be controlled, and the holes must be straight to accept a No. 6 rebar. The drilling angle must be accurate and repeatable, and the rate of drilling should be fast. To achieve these requirements, Kansas requires the use of a vacuum drill that sucks up dust through the center of the hollow drill bit. A proprietary trailer-mounted vacuum drill is available for less than $40,000. A truck-mounted drill is available, too, but it costs more.

A pachometer, which usually costs less than $3,000, is needed to locate the transverse bridge deck reinforcement. Without this instrument, too many carbide drill tips would be broken and much production time would be lost trying to locate holes by trial and error.

The epoxy injection pump must also meet certain specifications. The pump must be positive displacement and deliver a certified volume ratio of hardener to resin in the temperature and pressure range needed to perform the injection. It must be able to deliver a sustained pressure of 100 psi and must be controllable between 20 and

100 psi. The injection nozzle must lock in the hole and hold 100 psi without leaking. The nozzle is a device which the contractor can build himself or have a local machine shop build from a KsDOT design at minor cost. Epoxy components must be kept separate and mixed just ahead of the injection nozzle.

Stratton and Crumpton (1984) further report that an efficient crew can completely post-reinforce a 3-span bridge in less than one week.

Since 1981, the Kansas DOT has used post-reinforcement to repair over 20 bridges at approximately $1,000 per girder, as compared to approximately $40,000 per girder for girder removal and replacement.

It should be pointed out that while this procedure was specifically developed for use with bridge deck beams and girders, there appears to be no reason why the procedure cannot be modified for other applications.

PART IV: SUMMARY AND RECOMMENDATIONS

According to the results of a survey initiated in 1982 by the U.S. Army Corps of Engineers (McDonald and Campbell, 1985), the three most common types of deficiencies encountered in concrete hydraulic structures were (a) cracking, (b) seepage, and (c) spalling. These three general catagoreis of deficiencies accounted for 77% of the 10,096 deficiencies identified during a review of available inspection reports for the Corps' civil works structures. Concrete cracking was the deficiency most often observed, accounting for 38% of the total. _In situ_ repair procedures may not be readily applicable in the repair of seepage deficiencies; however, the problems normally resulting from deterioration due to cracking and spalling do seem to be suited to _in situ_ repair procedures.

A literature survey, private discussions, and responses to mail and telephone inquiries have disclosed a wide range of repair methods and materials currently available for the _in situ_ repair of cracked and spalled concrete. Crack repair methods include pressure injection, routing and sealing, stitching, addition of reinforcement, drilling and grouting, flexible sealing, grouting, drypack mortar, crack arrest, polymer impregnation, and overlays and surface treatments. Methods for repairing spalled concrete include coatings, concrete replacement, grinding, jacketing, shotcreting, prepacked concrete, and thin-bonded or unbonded overlays. Repair materials include bituminous materials, portland-cement concrete, mortar and grouts, epoxies, expandable mortars, grouts and concretes, linseed oil, latex-modified concrete, and polymer-concrete materials.

From an evaluation of the repair techniques and materials identified, five procedures (three crack repair techniques and two techniques for repairing spalled concrete) were identified as being the most applicable for _in situ_ repair of concrete hydraulic structures. The selected

techniques include pressure injection, polymer impregnation, and addition of reinforcement. In conjunction with these repair procedures, thin reinforced overlays and shotcrete can be used to repair spalled concrete surfaces as well as to resurface a cracked structure after it has been repaired.

Pressure injection, generally with epoxy adhesives, has been used extensively for about 25 years to repair a variety of concrete hydraulic structures. Consequently, much of the expertise, technology, materials, and equipment necessary for successful application of the process already exists. Advantages of pressure injection repair techniques include the following: the cracks are sealed both internally and externally; by proper selection of a water-compatible adhesive, cracks saturated with water can be repaired; pressure injection can be used, within limits, against a hydraulic head; and cracks as fine as 0.002 in. can be repaired.

Pressure injection as a repair technique has the following limitations: the process is generally restricted to the repair of members that have not yet begun to spall significantly and to the repair of dormant cracks; and the process may leave scars on the surface of the member where the cracks have been injected. Pressure injection, however, appears to be one of the most viable methods available for repairing severely cracked concrete structures.

Limited use has been made of polymer impregnation over the last 8 to 10 years to rehabilitate highly deteriorated concrete structures. Most of these applications have been experimental in nature, and routine applications are not yet common, most likely because the process is relatively new and uses specialized materials and equipment requiring a high level of expertise and supervision to ensure success.

Limitations of the polymer impregnation process include the following. The monomer systems currently in use require specialized safety procedures since they are considered flammable and toxic. In addition, they are not water compatible. Therefore, to obtain a complete cure of

the monomer system and to ensure adequate penetration of the monomer into the pore structure of the concrete, it is necessary to dry the concrete before impregnation in order to remove the free moisture within the pores.

Despite these limitations, polymer impregnation appears to be one of the best methods available for rehabilitating or improving the overall physical and mechanical properties of highly deteriorated low-quality or non-air-entrained concrete. Properties of polymer-impregnated concrete which make it very attractive for use in concrete hydraulic structures include low permeability to water and chloride penetration, high abrasion resistance, excellent durability during cycles of freezing and thawing, and compressive and flexural strengths three to four times greater than those of ordinary concrete.

The addition of reinforcement to deteriorated concrete structures, either internally or externally, has also been shown to be an excellent means for repairing cracked structures, particularly when it is also desirable to improve or restore the strength properties of the members being repaired. Both methods of repair have been used extensively for a number of years. As a result, much of the necessary materials, equipment and expertise necessary for the successful application of the repair techniques already exist.

Once the cracks in a member have been repaired, it may be necessary to resurface the member to (a) cover up any scars or imperfections left by the repair procedure, (b) repair minor damage resulting from spalling, or (c) provide the member with a more durable wearing surface. Thin reinforced overlays, either bonded or unbonded, and shotcrete appear to be the best methods available. Conventional portland-cement mortar systems or concrete-polymer material systems can be used for either resurfacing method.

Although each of the selected repair techniques has definite advantages over other forms of repair, it is felt that they could be improved to make them more applicable to the repair of hydraulic structures.

Areas which should be investigated include the following:

(a) Development of a repair technique which uses both pressure injection and polymer impregnation. Many of the older structures which are cracked contain concrete that is of low quality or is non-air entrained. While pressure injection repair techniques will effectively seal the crack network and rebond the concrete, the concrete is still susceptible to further cracking because the overall quality of the concrete itself remains unchanged. Impregnation of the concrete would improve the physical and mechanical properties of concrete, thereby reducing the possibility of further deterioration.

(b) Identification of water compatible monomers for use in impregnation work. At present, the monomers being used to impregnate concrete are not compatible with water. It is, therefore, necessary to dry the concrete to remove the free moisture from its pore system to ensure adequate penetration of the monomer. If this step could be eliminated, the time and cost of the repair process could be reduced. These monomers could also be used in pressure injection repair techniques provided a suitable initiator-promoter system can also be identified.

(c) Identification of low vapor pressure monomers for use in impregnation work. These monomers would help to eliminate some of the safety problems such as toxicity and flammability associated with the higher vapor pressure monomers currently being used.

(d) Develop and refine field impregnation techniques. Since most of the impregnation work performed to date has primarily been on horizontal surfaces, it will be necessary to develop impregnation techniques for use on curved and vertical surfaces such as those on concrete piers.

(e) Investigate the use of spray applicable polymer concrete overlays. Sprayable polymer concrete overlays can be used to resurface a concrete member once it has been repaired or to seal the concrete, thereby reducing its permeability. Fillers, such as calcined coke breeze, can be added to the overlay to make it electrically conductive. The overlay can then be used as the anode in an impressed current cathodic protection system to prevent reinforcing steel from corroding.

(f) Investigate the use of nonmetallic reinforcement in thin overlays. One problem with the use of thin overlays is that they are subject to reflective cracking. The use of nonmetallic reinforcing, such as polymer grids, may help to eliminate this problem.

(g) Work may be needed to adapt the Kansas Department of Transportation's post-reinforcement repair techniques to nonhorizontal applications.

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Table 1

Crack Repair Techniques for Concrete

Repair Technique	Type of Crack Dormant	Type of Crack Active	Comments
Pressure Injection	X		Little surface preparation is needed; scar marks may be left on surface where crack was injected. Limited to areas where concrete has not yet spalled. Structural quality bond is established but if large structural movements are still occurring, new cracks may open. Process can be used against a hydraulic head.
Routing and Sealing	X		Simplest method available for repair of cracks with no structural significance. Process not applicable to repair of cracks subjected to hydraulic head.
Stitching	X	X	Process will not close or seal cracks but can be used to prevent them from progressing. Generally used when it is necessary to reestablish tensile strength across crack.
Addition of Reinforcement	X	X	Primarily used to restore or upgrade structural properties of cracked members.
Drilling and Grouting	X		Technique applicable only when cracks run in straight line and are accessbile at one end.

Table 1 cont.

Crack Repair Techniques for Concrete

Repair Technique	Type of Crack Dormant	Type of Crack Active	Comments
Flexible Sealing	X	X	Technique is applicable where appearance is not important and in areas where cracks are not subjected to traffic or mechanical abuse.
Grouting	X		Wide cracks may be filled with portland-cement grout. Narrow cracks may be filled with chemical grouts.
Drypack Mortar	X		For use in cavities that are deeper than they are wide. Convenient for repair of vertical members.
Crack Arrest	X	X	Commonly used to prevent propagation of cracks into new concrete during construction.
Impregnation	X	X	Technique can be used to restore structural integrity of highly deteriorated or low quality concrete. Can be used to seal small crack networks.
Overlays and Surface Treatments	X	X	Slabs containing fine dormant cracks can be repaired using bonded overlays. Unbonded overlays should be used to cover active cracks.
Autogenous Healing	X		A natural process of crack repair has practical applications for closing dormant cracks in moist environments.

Table 2

Techniques for Repairing Spalled Concrete

Repair Technique	Comments
Coatings	This technique is generally used when the scaling or spalling is limited to a very thin region at the surface of the concrete.
Concrete Replacement	This technique is one of the most commonly used and is appropriate for applications where the cause of deterioration is nonrepeating or has been eliminated.
Grinding	This technique can be used when the deterioration is limited to a thin region at the surface of the concrete.
Jacketing	This technique entails fastening a material to the existing concrete that is more resistant to the environment that is causing the deterioration.
Shotcreting	This technique is practical for large jobs, on either vertical or horizontal surfaces, where the cavities are relatively shallow.
Prepacked Concrete	This technique is suitable for inaccessible applications, such as submerged concrete or deteriorated concrete that is being jacketed.
Thin-Bonded and Unbonded Overlays	Thin overlays are often used to repair surfaces that are basically sound structurally but have deteriorated because of cycles of freezing and thawing, heavy traffic, or other exposures which the original concrete was unable to withstand.

Table 3

Materials for Repairing Spalled Concrete

Repair Material	Comments
Bituminous Coatings	Asphalt- or coal-tar-based bituminous coatings are used to waterproof concrete or protect it, to some extent, from weathering.
Concrete, Mortar, or Grout	Portland-cement concrete, mortar, and grout have a number of advantages as a repair material, including: thermal properties similar to the existing concrete, similarity in appearance, comparatively low cost, availability, and familiarity.
Epoxies	Epoxies are most often employed in repair work for the following uses: as an adhesive to bond plastic concrete to hardened concrete or other rigid materials, for patching, and for coating concrete to protect it from aggressive environments.
Expanding Mortars, Grouts, and Concretes	These materials are generally proprietary materials to counteract the problem of shrinkage by incorporating ingredients which produce an expansive force approximately equal in magnitude to the shrinkage stresses.
Linseed Oil	Linseed oil is generally used to prevent or minimize additional scaling from occurring.
Latex-Modified Concrete	Latex-modified concretes have generally been used for resurfacing deteriorated floors and bridge decks. They typically develop higher strengths, bond better to existing concrete, have higher resistances to chloride penetration, and are more resistant to chemical attack than plain concrete.

Table 3 cont.

Materials for Repairing Spalled Concrete

Repair Material	Comments
Polymer Concrete	Polymer concrete has been used extensively to repair highway bridges and pavements. It has a number of advantages over normal concrete, including rapid curing characteristics, high early strength, good bond strength, and excellent durability through cycles of freezing and thawing.